R-94-04

VALLECITO DAM FLOW MEASUREMENT AND WEIR CALIBRATION

May 1994

U.S. DEPARTMENT OF THE INTERIOR
Bureau of Reclamation
Denver Office
Research and Laboratory Services Division
Hydraulics Branch

7-2090 (4-81)
Bureau of Reclamation

TECHNICAL REPORT STANDARD TITLE PAGE

1. REPORT NO. R-94-04	2. GOVERNMENT ACCESSION NO.	3. RECIPIENT'S CATALOG NO.

4. TITLE AND SUBTITLE Vallecito Dam Flow Measurement and Weir Calibration	5. REPORT DATE May 1994 6. PERFORMING ORGANIZATION CODE D-3751
7. AUTHOR(S) Roxanne George	8. PERFORMING ORGANIZATION REPORT NO. R-94-04
9. PERFORMING ORGANIZATION NAME AND ADDRESS Bureau of Reclamation Denver Office Denver CO 80225	10. WORK UNIT NO. 11. CONTRACT OR GRANT NO.
12. SPONSORING AGENCY NAME AND ADDRESS Same	13. TYPE OF REPORT AND PERIOD COVERED 14. SPONSORING AGENCY CODE DIBR

15. SUPPLEMENTARY NOTES

Microfiche and hard copy available at the Denver Office, Denver, Colorado

16. ABSTRACT

The Bureau of Reclamation used a 1:21 scale model in the Denver Office Hydraulic Laboratory to determine the best method of obtaining an accurate measurement of flows passing Vallecito Dam, both through the outlet works and over the spillway. Testing of the outlet works weir wall was performed to determine if, under a variety of flow conditions, the weir could be calibrated to accurately measure flows passing over the weir. Analyses determined that both the flow over the outlet works weir and the flow through the spillway radial gates can be accurately measured without any changes to the existing structures. For the outlet works weir, three possible equations for the discharge curve were obtained, each dependent upon the operating scenario in use. For the spillway radial gates, free flow discharge rating curves were calculated.

17. KEY WORDS AND DOCUMENT ANALYSIS

a. DESCRIPTORS-- hydraulic engineering/ hydraulic laboratories/ hydraulic models/ concrete structures/ diversion dams/ fish barriers/ stilling basins/water measurement or flow measurement/

b. IDENTIFIERS-- Vallecito Dam/ Pine River Project/ Colorado

c. COSATI Field/Group 13B ***COWRR:*** 1302 ***SRIM:***

18. DISTRIBUTION STATEMENT *Available from the National Technical Information Service, Operations Division, 5285 Port Royal Road, Springfield, Virginia 22161*	19. SECURITY CLASS (THIS REPORT) *UNCLASSIFIED* 20. SECURITY CLASS (THIS PAGE) *UNCLASSIFIED*	21. NO. OF PAGES 42 22. PRICE

R-94-04

VALLECITO DAM FLOW MEASUREMENT AND WEIR CALIBRATION

by

Roxanne George

Hydraulics Branch
Research and Laboratory Services Division
Denver Office
Denver, Colorado

May 1994

UNITED STATES DEPARTMENT OF THE INTERIOR ★ BUREAU OF RECLAMATION

ACKNOWLEDGMENTS

Don Fazzan from the Durango Projects Office provided assistance in coordinating the project and gathering historical project data. The studies were conducted under the direct supervision of Brent Mefford, Head, Hydraulic Structures Section, with general supervision and review from Philip Burgi, Chief, Hydraulics Branch.

U.S. Department of the Interior
Mission Statement

As the Nation's principal conservation agency, the Department of the Interior has responsibility for most of our nationally-owned public lands and natural resources. This includes fostering sound use of our land and water resources; protecting our fish, wildlife, and biological diversity; preserving the environmental and cultural values of our national parks and historical places; and providing for the enjoyment of life through outdoor recreation. The Department assesses our energy and mineral resources and works to ensure that their development is in the best interests of all our people by encouraging stewardship and citizen participation in their care. The Department also has a major responsibility for American Indian reservation communities and for people who live in island territories under U.S. administration.

CONTENTS

TABLES

FIGURES

CONTENTS — CONTINUED

FIGURES — CONTINUED

INTRODUCTION

Recent modifications to the Vallecito Dam outlet works structure have caused increased turbulence near the outlet works flow measurement device, making flow measurements unreliable. In addition, no accurate measure of flow through the spillway radial gates has ever existed. Consequently, the flow quantity passing Vallecito Dam is unknown.

The purpose of the Vallecito Study was to determine the best method of obtaining an accurate measurement of flows passing the dam, both through the outlet works and over the spillway.

CONCLUSIONS

Based on the analyses performed, flow over the outlet works weir can be measured within ±10 pct of the correct flow without any changes to the existing structures. An additional 2 to 3 pct improvement in the flow measurement accuracy can be attained by moving the location of the stilling basin wall piezometer tap closer to the weir.

For the outlet works weir, three possible equations for the discharge curve were obtained (table 7), each dependent upon the operating scenario in use. The equation for the most common operating scenario should be used to determine flow through the outlet works. For flows greater than 2,500 ft^3/s, the equations are less reliable, and actual discharges may vary from calculated discharges.

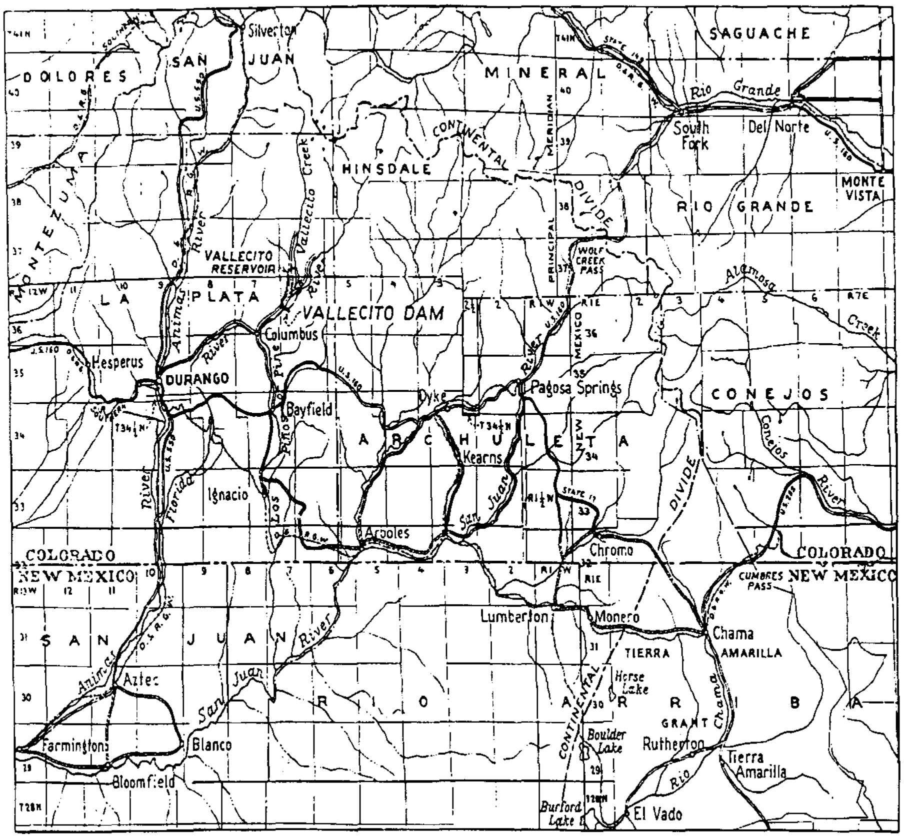
One option for obtaining better accuracy of measurement for the outlet works weir under all conditions would be to relocate the current stilling well tap (herein referred to as tap 1) about 27 ft downstream to the tap 3 (model) location and height. Test results indicated that flow near tap 3 was significantly less turbulent than flow near tap 1, and results more closely matched the existing calibrated curve under all operating scenarios.

Discharge rating tables for the spillway radial gates, including free and gated operation, were calculated based on model study data for uncontrolled flow at Vallecito Dam and gate controlled flow data for Granby Dam and Boysen Dam. Results are given in table 9. Based on comparisons of similar methods for evaluating radial gate discharge when applied to model study results of radial gate controlled spillways, a discharge measurement accuracy of about ±5 pct is expected for Vallecito spillway.

Canal radial gate algorithms given by Buyalski predicted free flow data for Vallecito reasonably well at low flows; however, predicted values deviated significantly at flows above about 5000 ft^3/s. This deviation from model free flow data can likely be attributed to the increasing effect of the spillway ogee crest as depth of flow increases.

BACKGROUND INFORMATION

Vallecito dam is a 162-ft-high zoned earthfill structure, located on the Pine River 18 miles northeast of Durango, Colorado (fig. 1). The dam was built by Reclamation in the period 1938-1941, primarily for flood control and irrigation purposes, but it also serves as a recreational site.



VICINITY MAP

SCALE OF MILES

Figure 1. - Location of Vallecito Dam.

The reservoir has an active capacity of 126,300 acre-ft, ranging from elevation 7582.5 to 7665.0. The spillway is a curved (super-elevated), concrete-lined, rectangular open channel, with a varying width of 50 to 125 ft (fig. 2). The spillway is about 2300 ft long, and has an elevation drop of about 124 ft. Flow through the spillway is regulated by three 37- by 19-ft radial gates located on the crest of the right abutment of the dam. The spillway was designed for a maximum flow of 33,000 ft^3/s.

The outlet works is a twin section concrete conduit which runs through the right abutment of the dam (figs. 3 and 4). Flow through the outlet works is regulated by two 5- by 5-ft slide gates, located below the right abutment, just upstream from the dam crest. The outlet works consist of two parallel 84-in-diameter horseshoe conduits, about 350 ft long, which pass below the dam and enter a 15-ft-wide, 400-ft-long rectangular concrete outlet channel located at the downstream toe of the right abutment. Flow from the outlet channel enters a baffled stilling basin, crosses over a weir wall at an angle of about 45°, and drops into the spillway channel. The outlet works was designed for a maximum flow of 3000 ft^3/s.

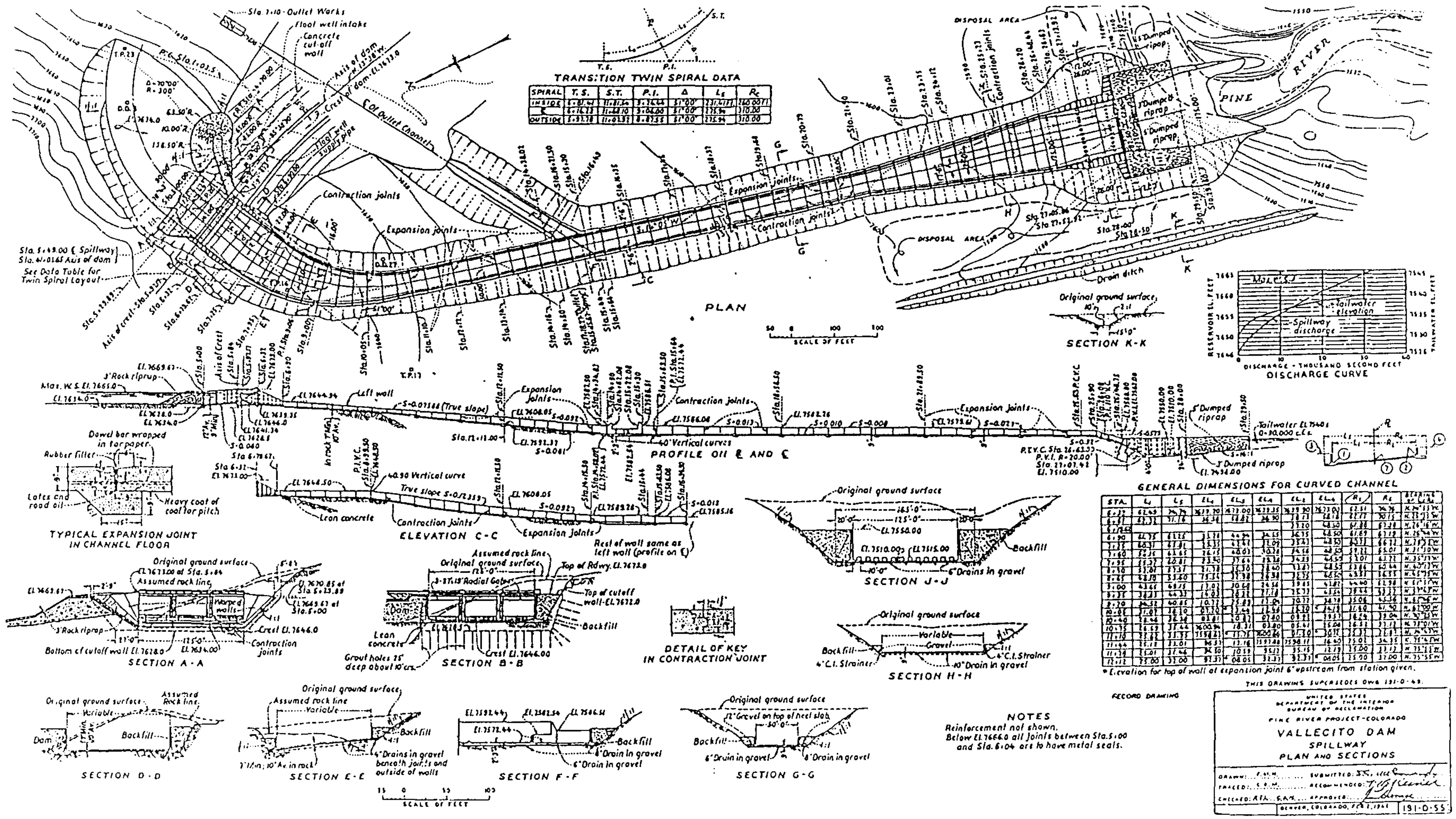


Figure 2. - Spillway plan and section.

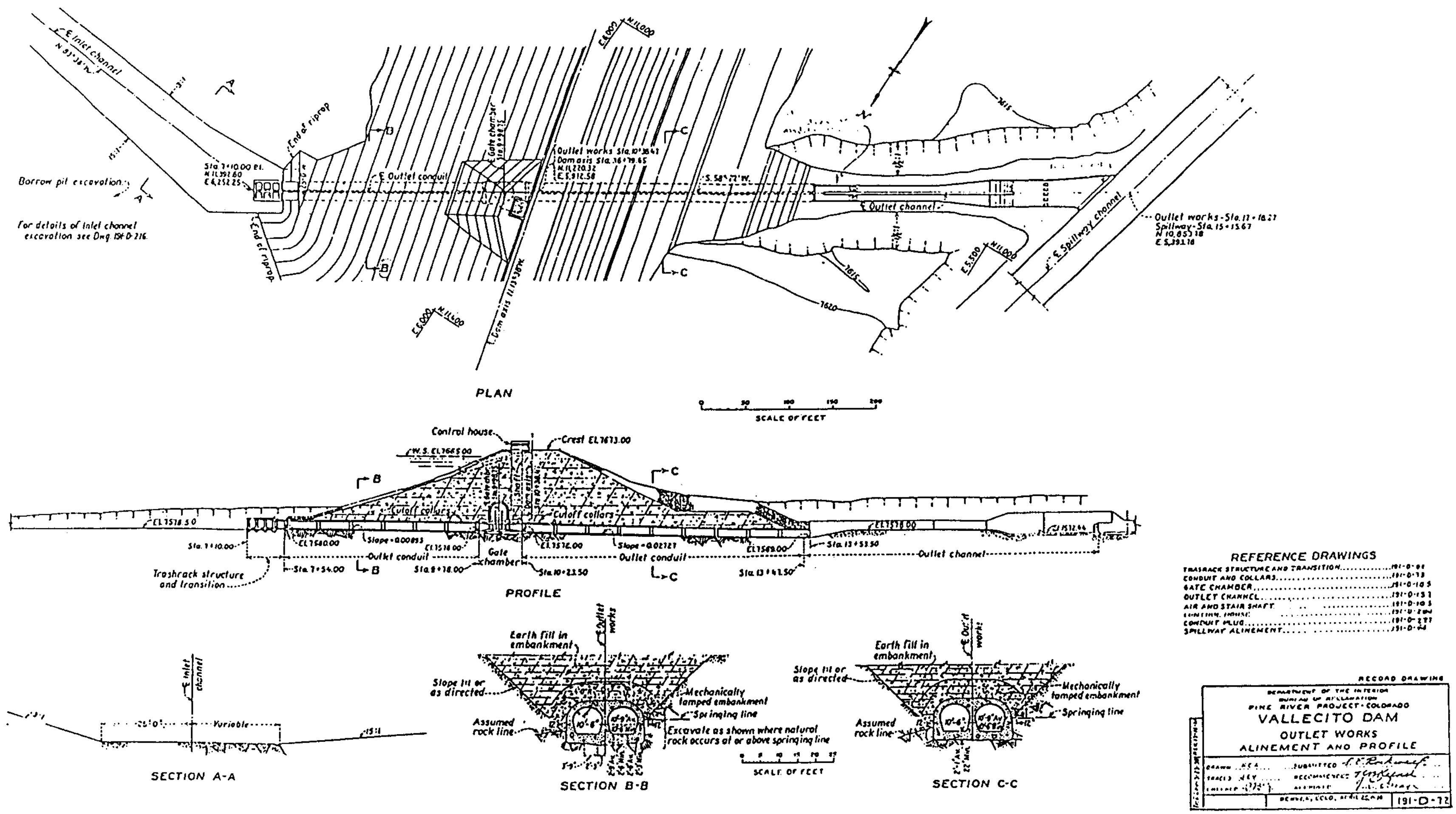


Figure 3. - Outlet works alignment and profile.

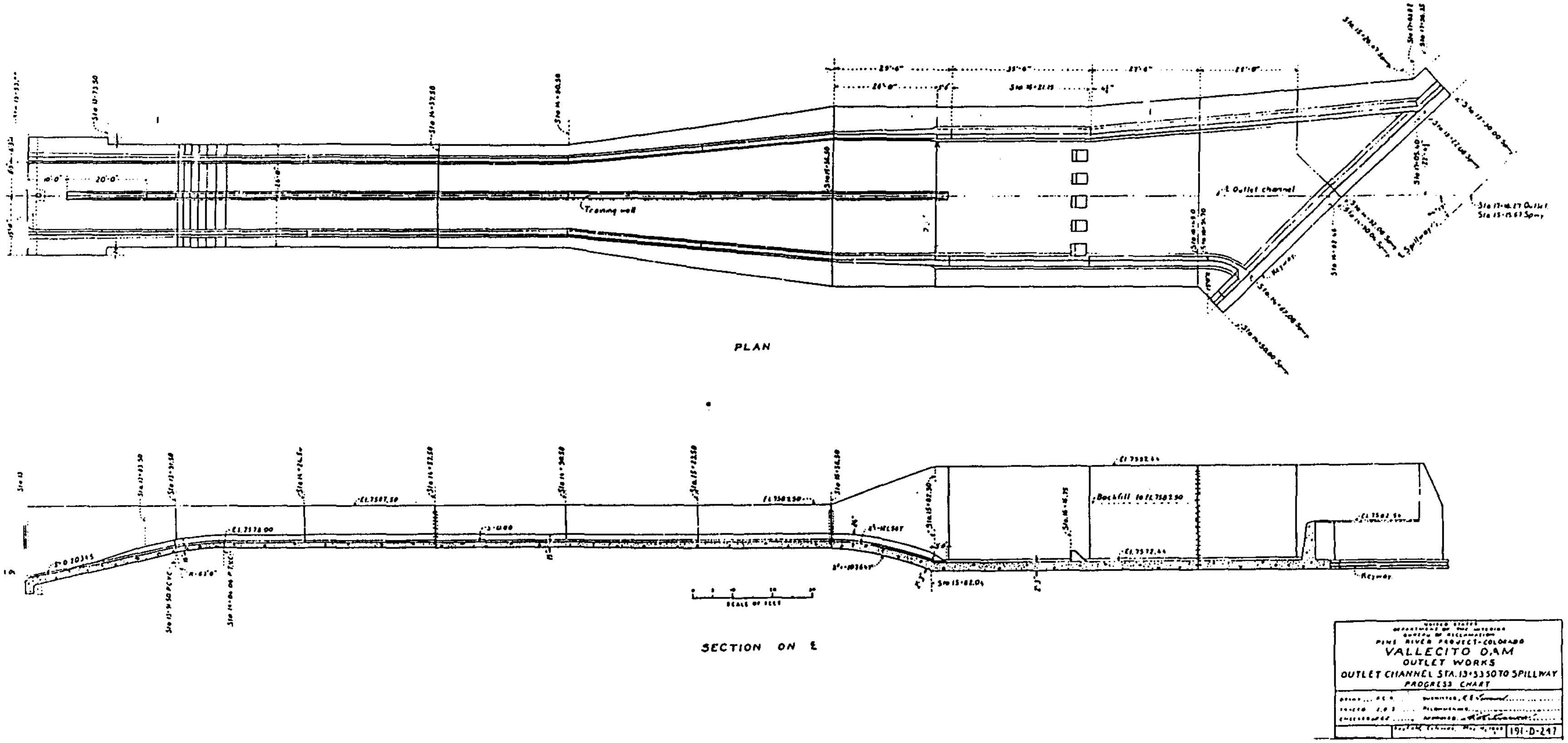


Figure 4. - Outlet works plan and profile.

In 1989, the left conduit of the outlet works was extended some 40 ft into the outlet channel, and the flow from this conduit is now being diverted via an 84-in butterfly valve to a privately owned powerplant located downstream from the spillway. This modification to the outlet works has created uncertainty as to the weir calibration because of a significant increase in the amount of turbulence and unsymmetrical flow patterns in the outlet channel. In addition, no spillway discharge calibration exists for gated control. Two separate tasks were undertaken to resolve these reservoir release questions at Vallecito Dam.

SCOPE OF WORK

To determine the amount of flow passing through the outlet works, a 1:21 scale (model:prototype) model of the outlet works basin and a downstream portion of the spillway channel was built in Reclamation's Hydraulic Laboratory. Testing of the outlet works weir wall was performed to determine if, under a variety of possible flow conditions, the weir could be calibrated to accurately measure flows passing over it. If the weir wall could not accurately be calibrated, then control points along the spillway were to be analyzed as possible water measurement locations.

Two methods were investigated to estimate discharge through the spillway radial gates. First, a numerical calibration using methods developed by Buyalski (1983) and a computer program written based on Buyalski's work were used. Buyalski's algorithms were designed for canal radial gates, but were assessed for their usefulness in conjunction with spillway radial gates. The second method investigated was based on work by Pomeroy (1942) and, later, Rhone (1959), in which comparisons of discharge coefficients for radial gate controlled spillways derived from model tests were studied.

OUTLET WORKS MODEL STUDY

Model Description

A similitude analysis was performed for the sections to be modeled, and a 1:21 scale model was determined to best meet the requirements for available material sizes, lab space, and Froude similitude. The model consisted of the entire outlet works basin and the downstream portion of the spillway channel, beginning just upstream from the outlet works intersection (fig. 5).

Reservoir head conditions for the outlet works were simulated using a pressure tank attached to a 10-hp pump regulated by a control valve. Flow into the outlet works was measured using a Controlotron Uniflow Universal Dual Path Transit Time Flow Meter (expected accuracy of ±2 pct). All discharge data for the weir calibration are based on the Controlotron meter.

Flow through the right conduit of the outlet works was regulated with a 4-in slide gate attached directly to the pressure tank. Horseshoe-shaped piping (3-ft length) was placed between the gate and the beginning of the outlet works basin to allow the flow to stabilize.

Flow through the left conduit of the outlet works was regulated with a 4-in butterfly valve, located 1 ft-5 in. downstream from the beginning of the outlet works basin (station 13+53.50 prototype). No slide gate was installed on the left conduit because flow through this conduit is currently being regulated in the field by the butterfly valve, and the slide gate remains open at all times.

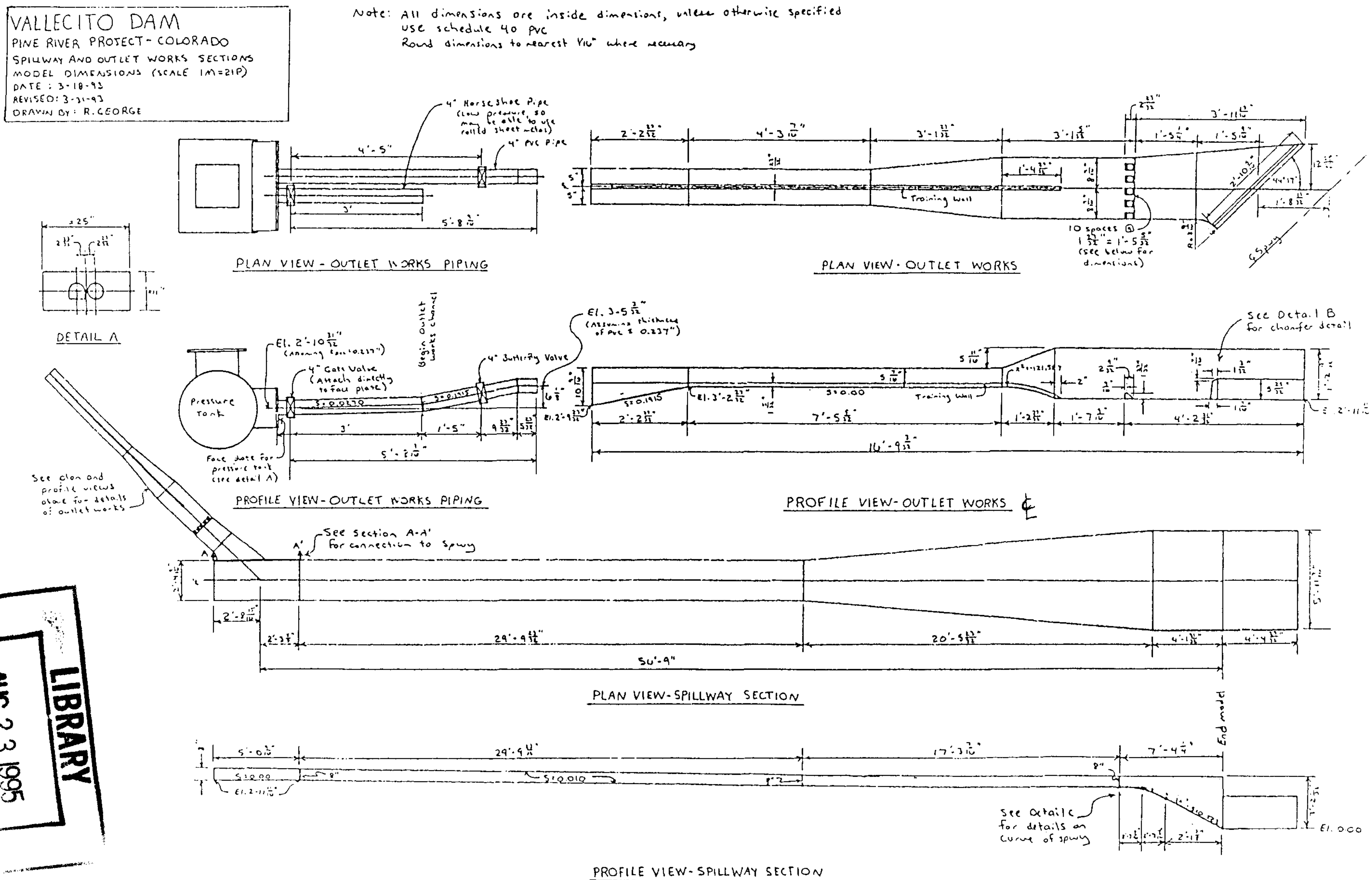


Figure 5. - Spillway and outlet works - model dimensions.

Head measurements at the weir were taken at three locations as shown in table 1. The location of taps 1 and 2 corresponds to locations of two of the taps in the prototype. Tap 3 was added in the model after initial tests indicated that flow in this location was less turbulent. Location of water surface elevations were measured at tap 1 using a 4-in stilling well, and at taps 2 and 3 using both piezometers and static pressure transducers.

Table 1. - Location of piezometer taps.

Tap No.	Location of tap (proto)	Elevation of tap (proto)
1	48 ft upstream from left inside corner of weir wall	9 ft above floor
2	48 ft upstream from left inside corner of weir wall	2 ft above floor
3*	21 ft upstream from left inside corner of weir wall	2 ft above floor

* This tap exists in the model only, and was added to compare fluctuation in head at different tap locations.

The model was designed to duplicate the prototype as closely as possible; however, some minor changes were made to the model design in the interest of time and cost:

- The channel floor of the outlet works basin near the inlet (stations 13+91 to 14+04 prototype) was modified from a curved slope to a straight slope, intersecting with the horizontal floor at station 14+00.5 (fig. 4).

- The fillets located along the bottom of the outlet works basin walls were not included in the model.

These small simplifications in the model will not affect the accuracy of the weir flow measurement.

Flow Profile Analysis

The outlet works basin is a "bump" type basin, designed to maintain submerged conditions at the tunnel outlets. Super-critical flow enters the basin, rises to a horizontal floor, and then expands and drops into a stilling basin. The stilling basin contains baffle blocks designed to dissipate energy by creating a hydraulic jump, thus forcing the flow to be subcritical prior to passing over the weir wall at the end of the basin. This type of design is common in Reclamation dams; however, some peculiarities at Vallecito Dam make flow measurement through the outlet works difficult to determine.

As mentioned previously, the addition of the butterfly valve in the left channel has altered the symmetry of the outlet channel, causing the flow to oscillate from side to side relative to the openings of both the butterfly valve and the right slide gate. In addition, the weir wall is at an angle to the flow of about 45°. Both of these conditions affect the location and shape of the hydraulic jump, which occurs just upstream from the measurement locations at higher flows.

Another peculiarity of the structure is that the weir crest is not a typical design. The weir crest has a very short length, similar to a sharp-crested weir, except that the sharp lip of the weir is located on the downstream side of the weir, similar to a broad-crested weir (fig. 6). Therefore, an analysis of flow passing over the weir was not possible. An original discharge curve based on tap 1 exists for the outlet works. However, no information is available as to how this curve was obtained. The extent to which the addition of the butterfly valve has changed the flow profile near the weir is also unclear.

The spillway channel flow profile was also analyzed to determine if control points existed which could be used to measure total flow for both the outlet works and spillway channel. Flow along the spillway channel was determined to be super-critical throughout the entire reach, which would make flow measurement difficult. Therefore, this option was not explored further.

Description of Tests Performed

Under normal operating conditions, all flow passing the dam is diverted to the powerplant. During high reservoir conditions, excess flows are passed either through the spillway radial gates or through the right slide gate of the outlet works. The butterfly valve (left side outlet) is opened only to release excess flow or to divert flow from the powerplant during down times. Typically, flow diverted to the powerplant ranges from 36 to 750 ft^3/s. During high reservoir conditions, excess flow passing through the right slide gate has been as much as 1800 ft^3/s.

To determine the accuracy of the weir under varying flow conditions, seven possible operating scenarios were tested, and are listed in table 2. These scenarios range from worst-case highly unsymmetrical flow conditions (trials 1-4), to best-case symmetrical flow conditions (trials 5-7).

Table 2. - Settings for control devices.

Trial	Butterfly valve	Slide gate
1	Closed	Slide gate opening varied, reservoir elevation held constant at 7665.0
2	Open 30°	Closed
3	Open 60°	Closed
4	Open 90°	Closed
5	Open 30°	Open 34%
6	Open 60°	Open 75%
7	Open 90°	Open 100%

Trial 1, in which the butterfly valve is closed and the right slide gate is set to release excess flow, is the most typical scenario in the field. The original outlet works discharge rating curves were used to determine the slide gate openings for a range of reservoir elevations and operating conditions. The reservoir elevation can vary about 85 ft to a maximum reservoir elevation of 7665.0.

Trials 2, 3, and 4, in which the right slide gate is closed and the butterfly valve is open 30°, 60°, and 90°, respectively, could occur if the powerplant were to shut down and flows being diverted to the plant were released through the outlet works.

Trials 5, 6, and 7, in which the slide gate and butterfly valve are opened to approximate equal flows, is the best possible operating scenario because flow is nearly symmetrical down the outlet works channel. A coefficient of discharge curve was not available for the prototype butterfly valve; therefore, discharges were estimated using Corp of Engineers Hydraulic Design Chart 331-1 (fig. 7).

The maximum design flow for the outlet works is 3000 ft^3/s, although this value is seldom reached. However, to calibrate the weir over all possible flow ranges, data were collected for the entire range using increments of 200 ft^3/s to 400 ft^3/s. For each flow increment, water surface elevations at each weir tap were recorded, as well as the approximate jump location.

Two sets of data were collected for each trial. Initially, water surface elevations were measured visually using piezometer tubes. At higher flows, however, a large fluctuation in water surface elevation occurred, and visual inspection became difficult. Therefore, pressure transducers were added to taps 1 and 3 and then attached to a statistical voltmeter, which averaged several values for each tap over an interval of about 30 seconds. After inspection of the first data set, little to no differences were found between data from tap 2 and those from tap 1. Therefore, to simplify data reduction, data from tap 2 were averaged with data from tap 1 for the first data set, and no values from tap 2 were taken during the second data set.

Analysis and Results

The general equation for critical flow over a weir can be derived from the energy equation as:

$$Q = C_d * b * h^{1.5} \tag{1}$$

where: Q = discharge
C_d = coefficient of discharge
b = width of weir perpendicular to flow
h = depth of flow upstream from the weir crest

For standard weir types, the coefficient of discharge is well documented in the literature. However, the Vallecito outlet works weir crest is non-standard in that the crest shape deviates from a standard sharp-crest or broad-crest design. Boss (1989) refers to the Vallecito weir as a short-crested weir. This class of weirs often performs as broad-crested weirs under low heads and sharp-crested weirs under high heads.

A standard, broad-crested weir requires the existence of a hydrostatic pressure distribution on the crest. This condition has been found to exist when the ratio of head (h) to crest width (L) in the direction of flow, h/L, lies between 0.07 and 0.50. For the Vallecito weir, this condition means that the coefficient for a broad-crested weir is only applicable for depths of flow between 9/32 to 2 in (prototype), which correspond to discharges of 0.6 to 12.3 ft^3/s, respectively.

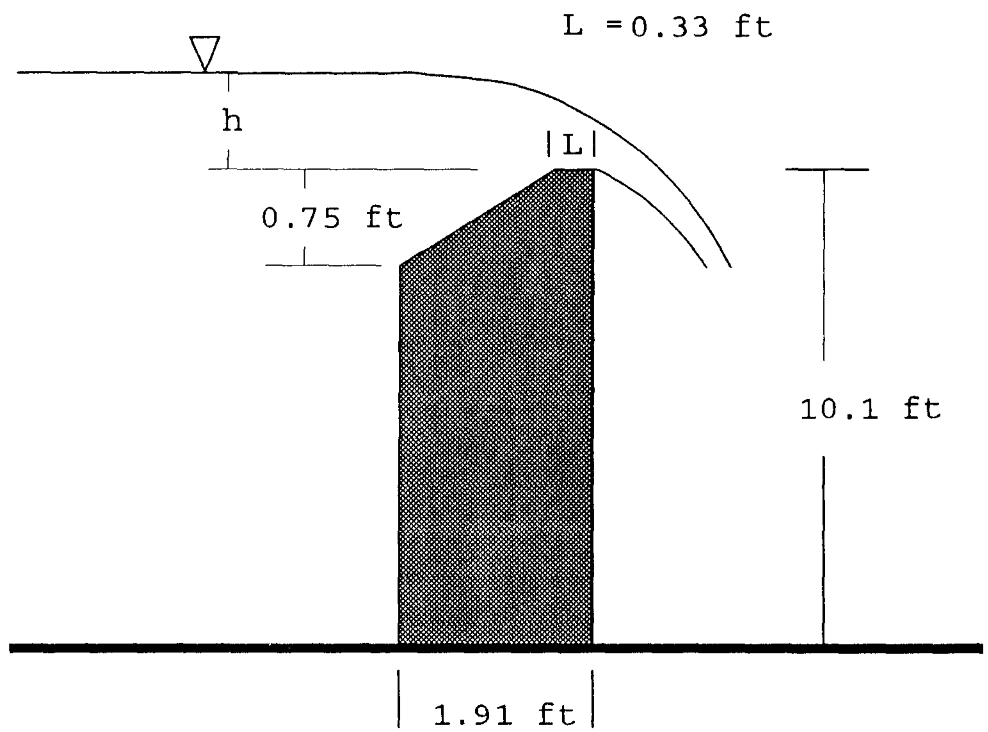
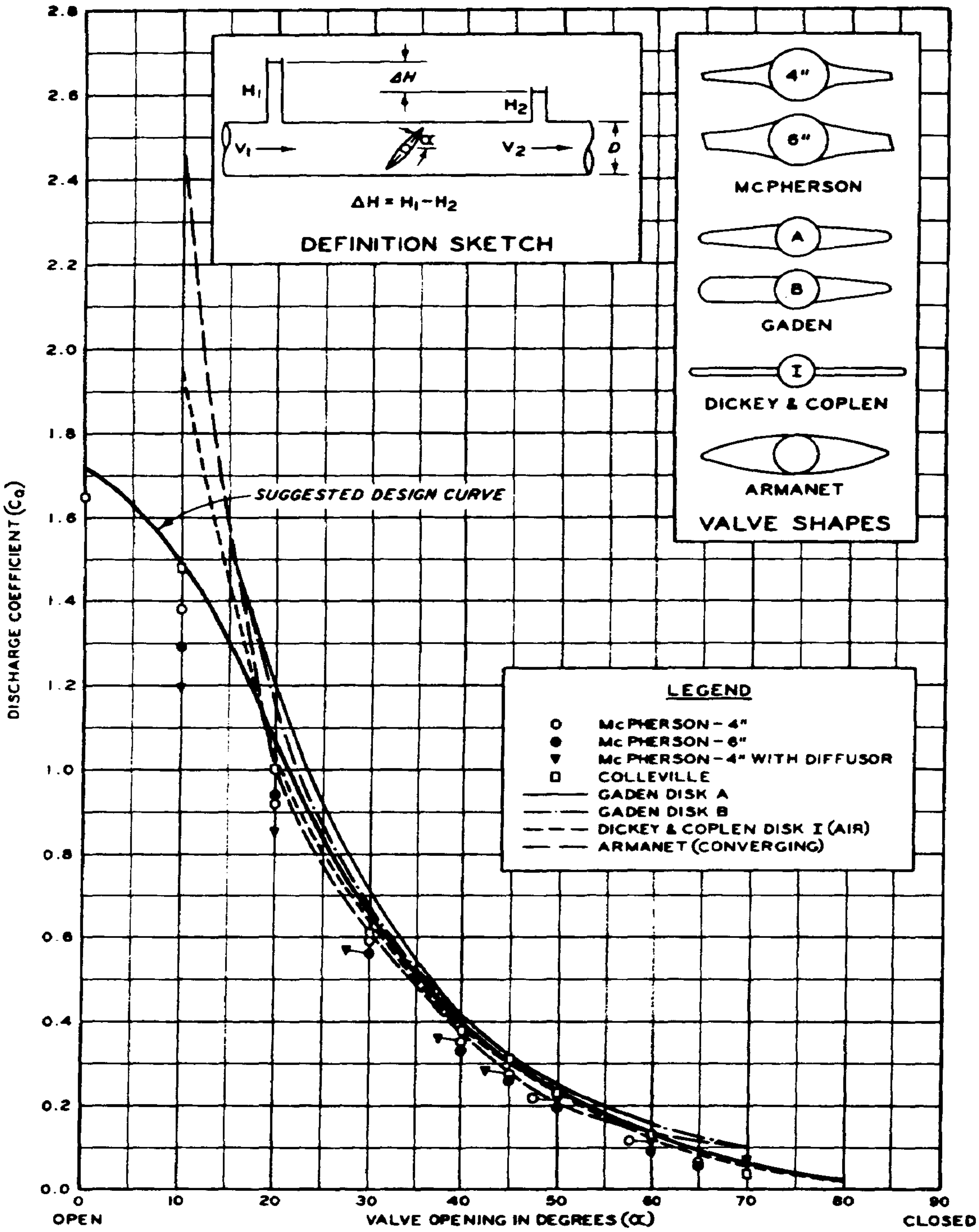


Figure 6. - Vallecito outlet works weir.



BASIC EQUATION

$$Q = C_Q D^2 \sqrt{g} \sqrt{\Delta H}$$

WHERE:

Q = DISCHARGE IN CFS

C_Q = DISCHARGE COEFFICIENT

D = VALVE DIAMETER IN FT

g = GRAVITY CONSTANT = 32.2 FT/SEC^2

ΔH = PRESSURE DROP ACROSS THE VALVE IN FT OF WATER

BUTTERFLY VALVES

DISCHARGE COEFFICIENTS
VALVE IN PIPE

HYDRAULIC DESIGN CHART 331-1

WES 8-58

Figure 7. - Discharge coefficients for butterfly valves.

The equation for flow over a sharp-crested weir is given by:

$$Q=C_e*2/3*(2*g)^{0.5}*b*h^{1.5} \tag{2}$$

where: C_e = 0.602+0.075(h/p)
g = gravitational constant
p = height of the weir

Substituting in the known values for the Vallecito weir reduces the above equation to the following:

$$Q=C_e*321*h^{1.5} \tag{3}$$

where: C_e = 0.602+0.075(h/10.1) (C_e is always less than 1.0)

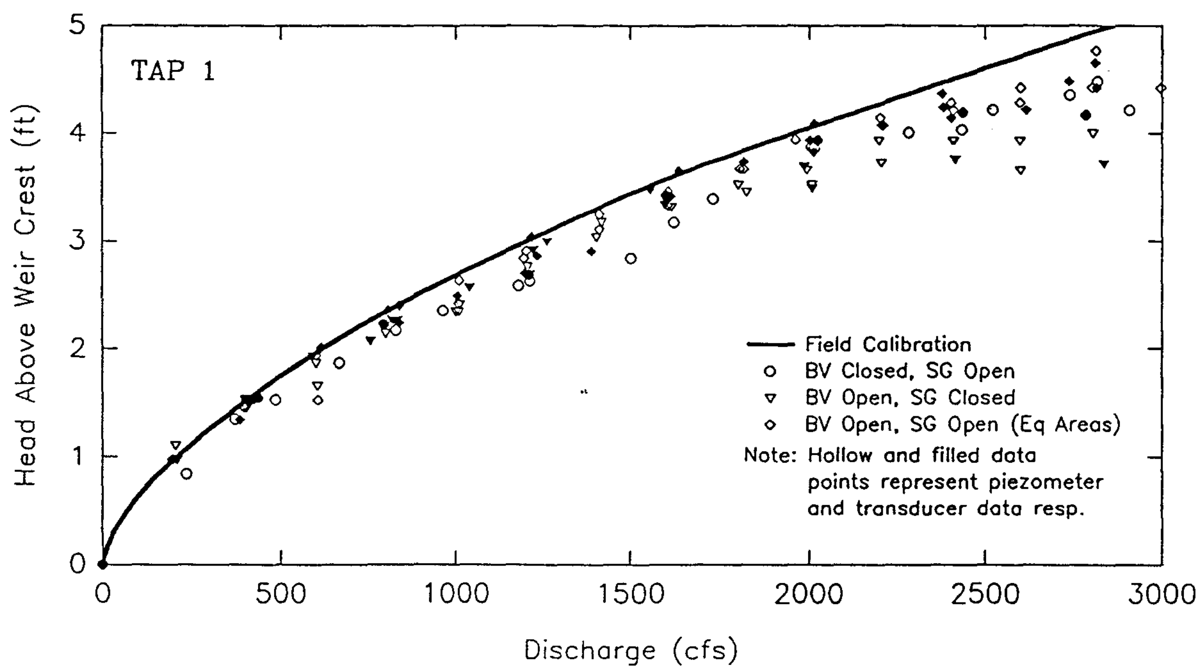
Limitations for a standard sharp-crested weir require a ratio of h/L greater than 15. For the Vallecito weir, the length of the weir crest is 0.33 ft, which means that the depth of flow over the weir must be greater than 5 ft for this equation to apply. Therefore, the Vallecito weir acts as a sharp-crested weir for heads greater than 5 ft or flows exceeding 2,294 ft^3/s.

A non-standard flow measurement device like the Vallecito weir requires a hydraulic model study to determine calibration and performance information over the full discharge range. During the model tests, the measured discharge and the head above the weir crest for each trial run were recorded and analyzed. Data for each pressure tap were analyzed separately, and best fit curves were found for each trial. Best fit curve equations of the form $Q=A*h^B$ and $Q=A*h^{1.5}$ were used as models because these equations are the general form for flow over a weir.

After analyzing the model data, the coefficients for the equation $Q=A*h^{1.5}$ were found to consistently plot below the data for low flows and above the data for higher flows, indicating that a higher exponential term would be more valid. The equation $Q=A*h^B$ fit the data much better throughout the range of flows, and was determined to be the better fit for all trials.

Tap 1 plots appear on figures 8 to 11, and tap 3 plots appear on figures 12 to 15. Figures 8 and 12 show all data points plotted against the existing field calibration curve for taps 1 and 3, respectively. Figures 9 to 11 and 13 to 15 show the data broken down into the three possible operating situations discussed previously for taps 1 and 3 respectively. Tables 3 and 4 summarize the best fit equations for taps 1 and 3 respectively, for each data set, and table 7 summarizes the best fit equations for each tap for combined data sets. The equation for the existing field calibration curve appears below these tables. Tables 5, 6, and 8 summarize the percent variance in discharge for each tap compared to the existing calibration curve.

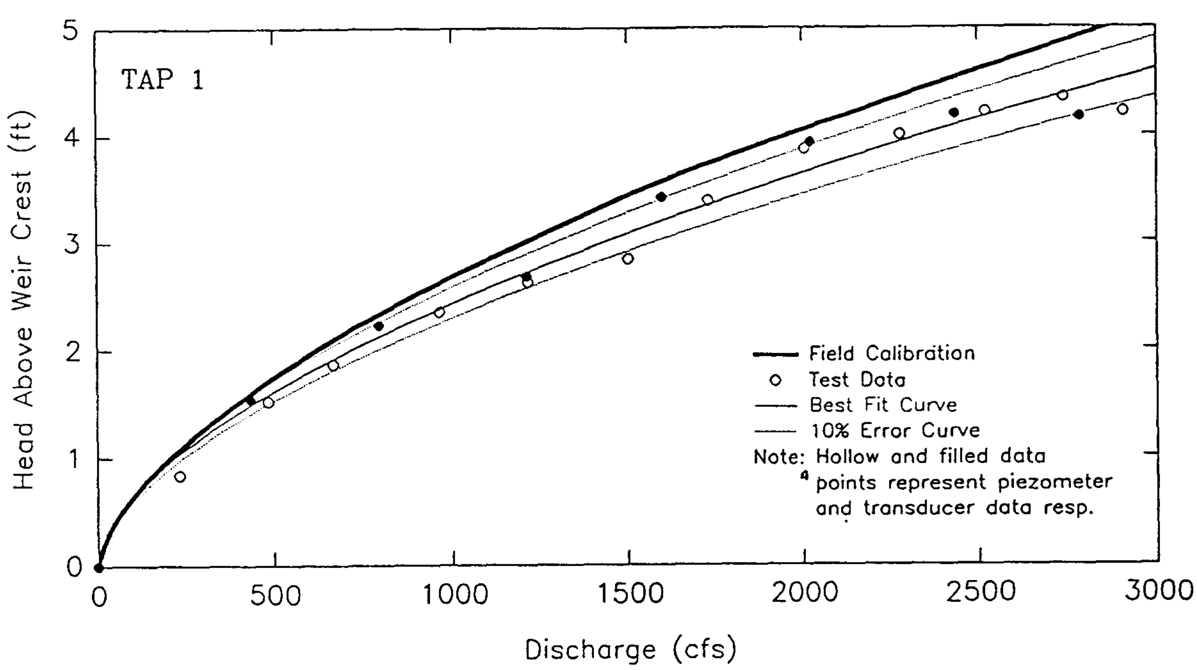
VALLECITO DAM – WEIR CALIBRATION
Field Calibration vs Collected Data



Summary of All Data Collected

Figure 8. - Summary of all head versus discharge data collected - tap 1.

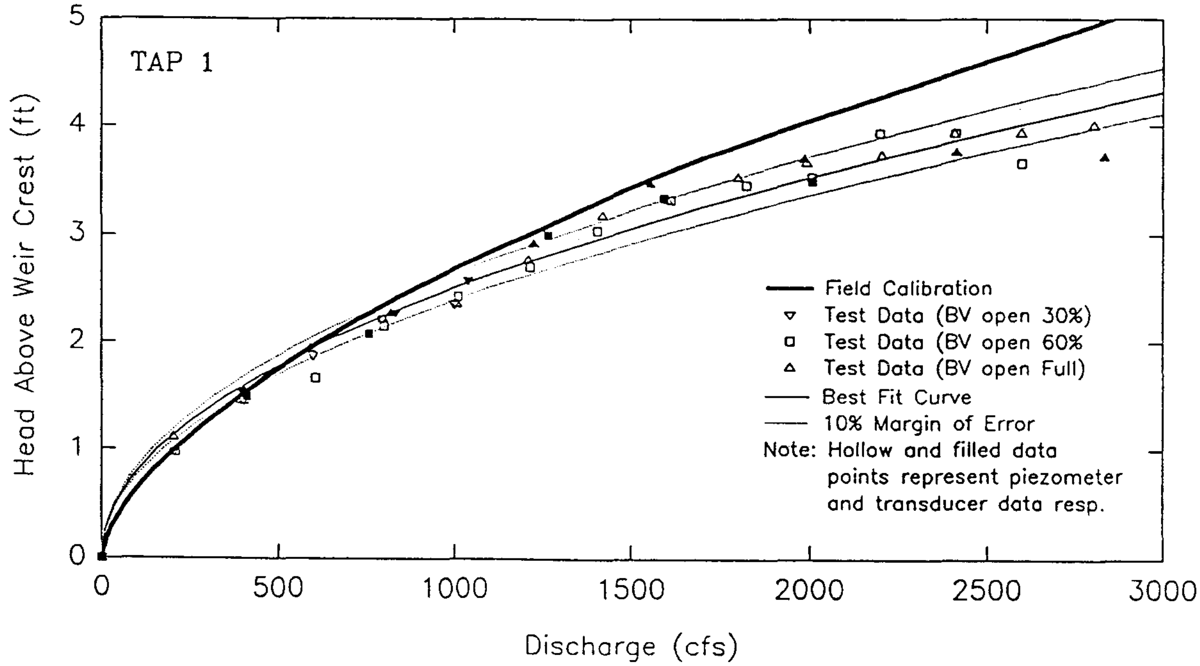
VALLECITO DAM – WEIR CALIBRATION
Field Calibration vs Collected Data



Trial 1: Butterfly Valve Closed, Slide Gate Open

Figure 9. - Head versus discharge data - tap 1, trial 1.

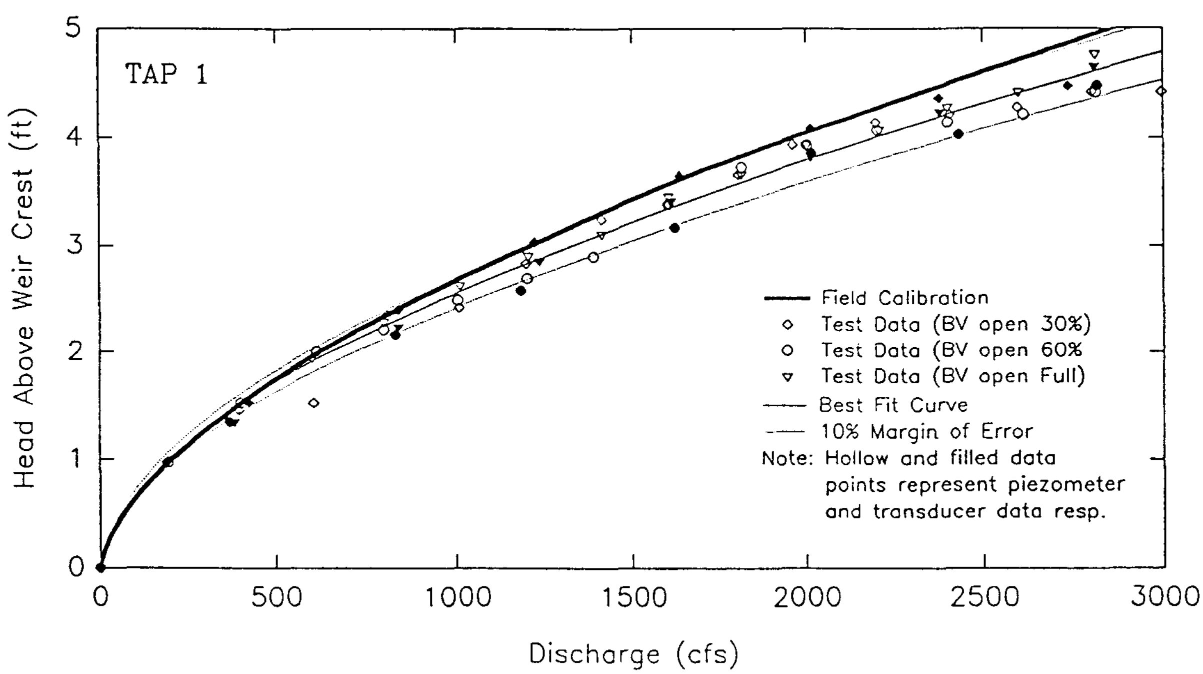
VALLECITO DAM – WEIR CALIBRATION
Field Calibration vs Collected Data



Trials 2,3,4: Butterfly Valve Open, Slide Gate Closed

Figure 10. - Head versus discharge data - tap 1, trials 2, 3, and 4.

VALLECITO DAM – WEIR CALIBRATION
Field Calibration vs Collected Data



Trials 5,6,7: Butterfly Valve Open & Slide Gate Open Equal Areas

Figure 11. - Head versus discharge data - tap 1, trials 5, 6, and 7.

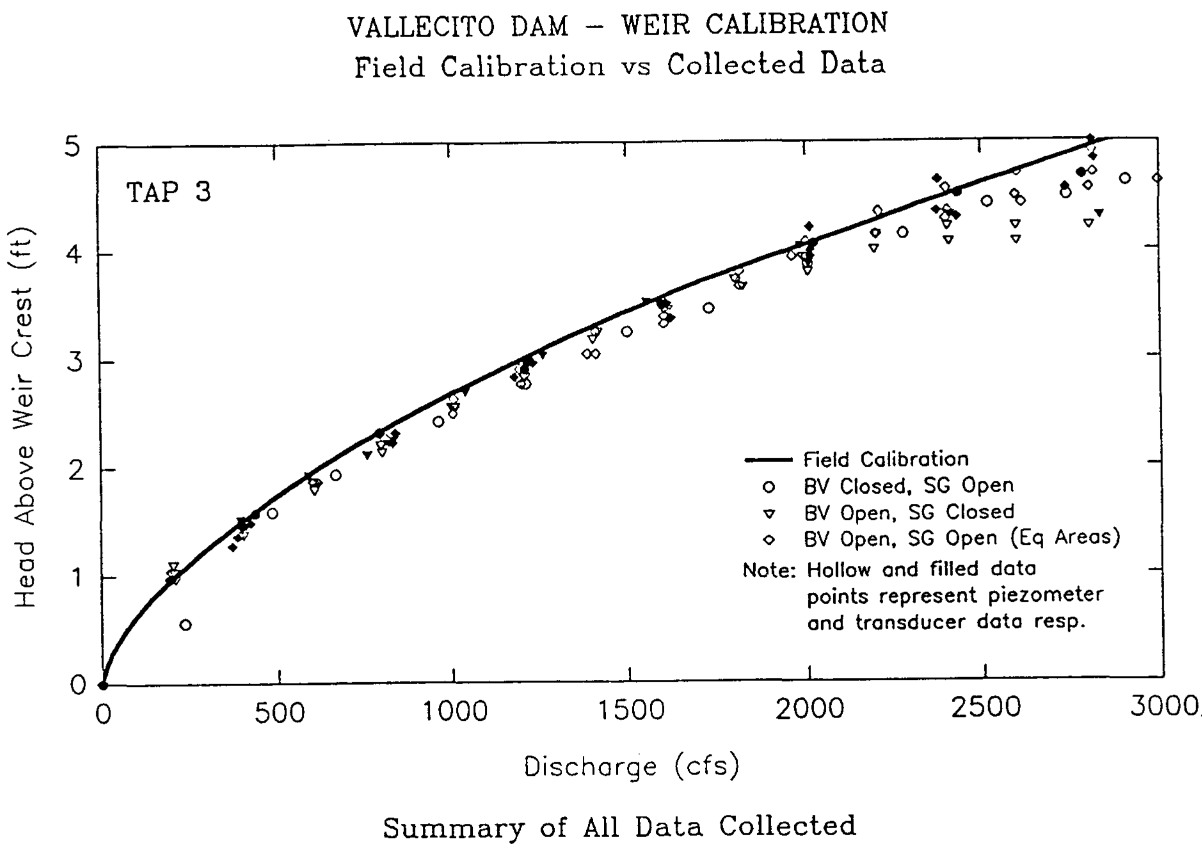


Figure 12. - Summary of all head versus discharge data collected - tap 3.

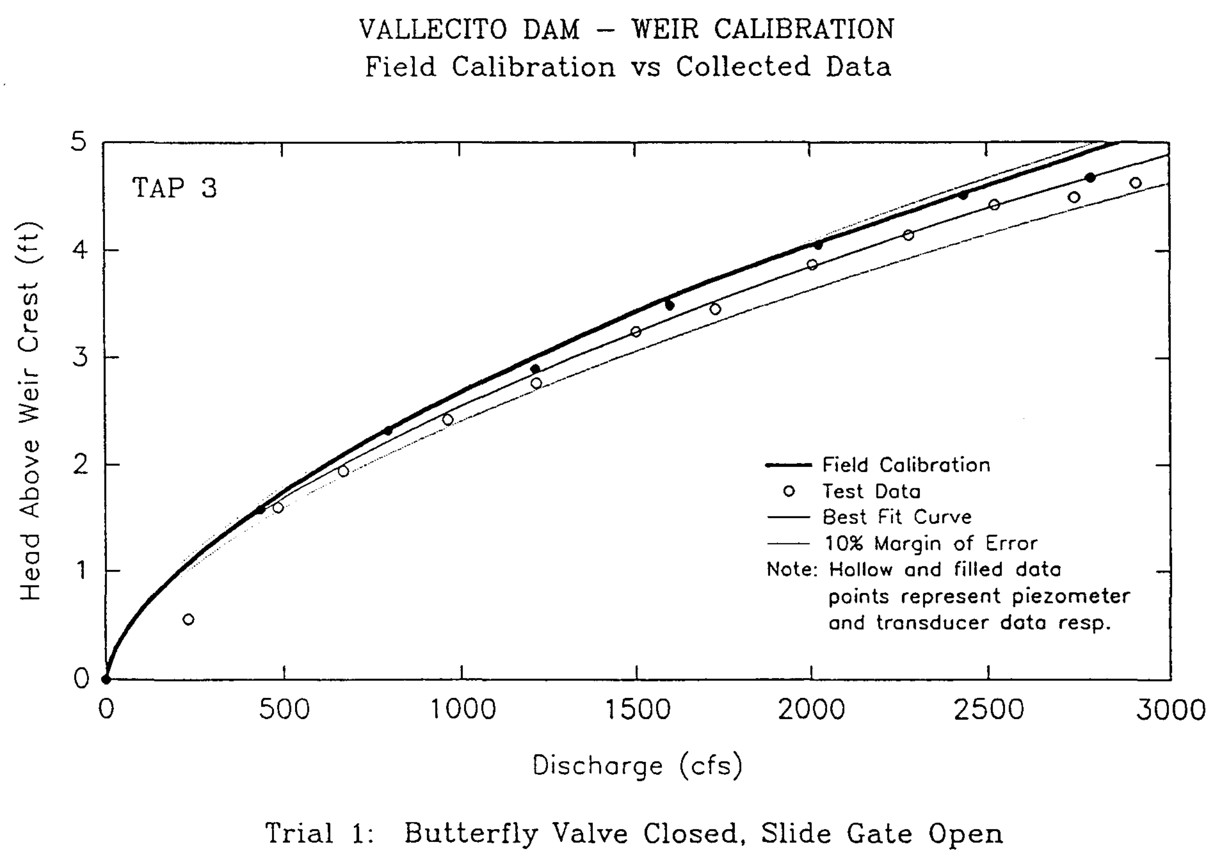


Figure 13. - Head versus discharge data - tap 3, trial 1.

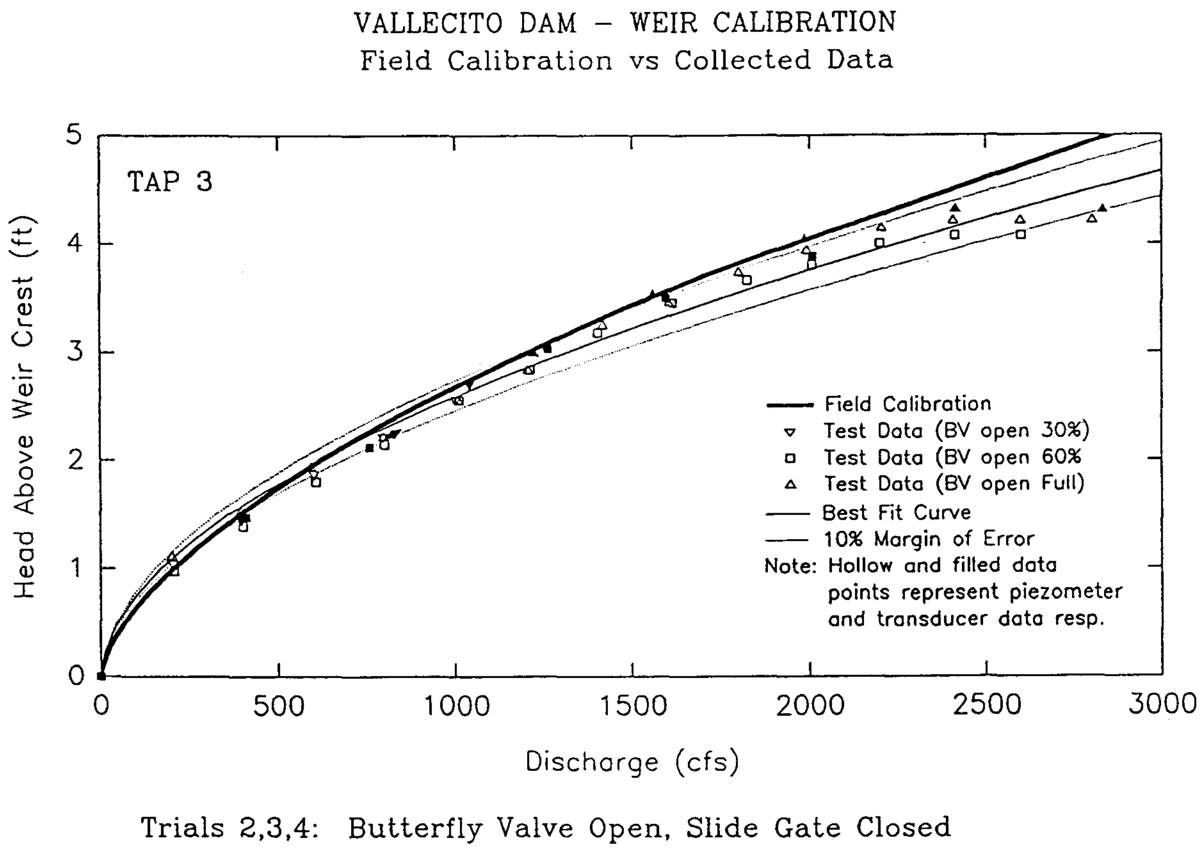


Figure 14. - Head versus discharge data - tap 3, trials 2, 3, and 4.

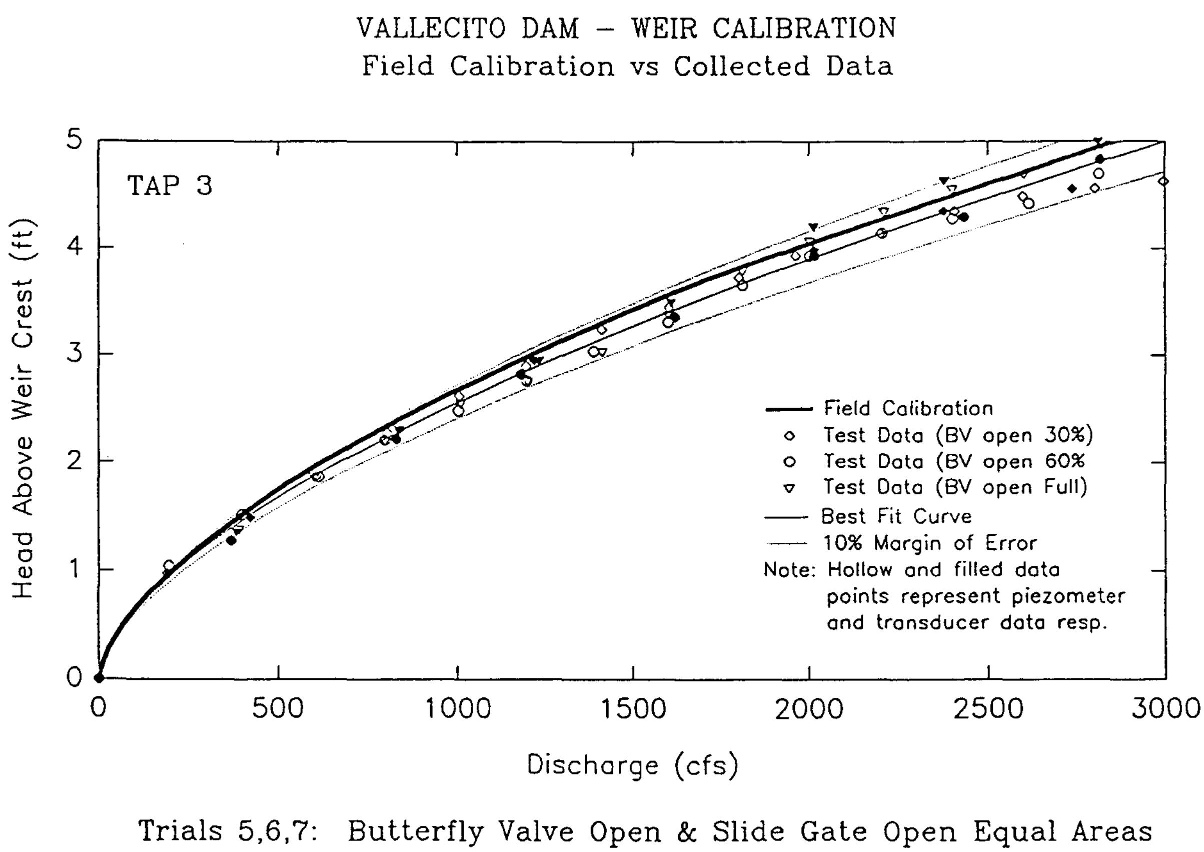


Figure 15. - Head versus discharge data - tap 3, trials 5, 6, and 7.

Table 3. - Summary of best fit curves - tap 1.

$Q=A*H^B$						
Trial	Data set 1			Data set 2		
*****	A	B	r^2	A	B	r^2
1	242.57	1.65	0.98	180.57	1.84	0.97
2	192.94	1.86	0.99	180.51	1.86	1.00
3	207.06	1.79	0.96	175.76	1.88	0.98
4	121.98	2.22	0.98	94.02	2.44	0.92
5	169.07	1.86	0.98	148.29	1.90	0.99
6	186.14	1.80	0.99	238.16	1.64	0.99
7	188.18	1.75	1.00	213.24	1.67	1.00
FIELD	191.41	1.68	1.00	NA	NA	NA

r^2 = coefficient of determination, i.e., variance of data from the best fit curve - the closer to 1.0, the better the fit.

Table 4. - Summary of best fit curves - tap 3.

$Q=A*H^B$						
Trial	Data set 1			Data set 2		
*****	A	B	r^2	A	B	r^2
1	215.53	1.68	0.99	185.72	1.73	1.00
2	197.10	1.74	1.00	237.02	1.50	1.00
3	181.40	1.82	0.99	208.84	1.65	1.00
4	141.99	2.01	0.96	149.62	1.93	0.97
5	161.64	1.86	0.99	187.21	1.74	1.00
6	202.69	1.71	1.00	232.69	1.59	1.00
7	230.62	1.57	0.99	227.69	1.54	1.00
FIELD	191.41	1.68	1.00	NA	NA	NA

r^2 = coefficient of determination, i.e., variance of data from the best fit curve - the closer to 1.0, the better the fit.

Table 5. - Comparison of field calibration to data - tap 1.

Field cal $Q(ft^3/s)$ /trial	Tap 1: % Variance in Q vs. field calibration*						
	1	2	3	4	5	6	7
500	+16	+10	+10	-15	-4	+10	+5
1000	+18	+15	+18	+7	+3	+13	+7
1500	+19	+18	+23	+23	+8	+15	+8
2000	+20	+20	+26	+36	+11	+16	+9
2500	+20	+22	+29	+47	+14	+17	+10
3000	+21	+23	+31	+57	+16	+18	+10

* "+" indicates actual flow is greater than field calibration.
"-" indicates actual flow is less than field calibration.

Table 6. - Comparison of field calibration to data - tap 3.

Field cal $Q(ft^3/s)$ /trial	Tap 3: % Variance in Q vs field calibration*						
	1	2	3	4	5	6	7
500	+8	+8	+2	-9	-4	+10	+12
1000	+9	+6	+8	+2	+2	+10	+7
1500	+9	+5	+11	+10	+6	+10	+4
2000	+9	+4	+14	+15	+8	+9	+2
2500	+9	+4	+16	+20	+11	+9	0
3000	+9	+3	+17	+24	+12	+9	-1

* "+" indicates actual flow is greater than field calibration.
"-" indicates actual flow is less than field calibration.

Table 7. - Summary of best fit curves - combined data.

$Q=A*H^B$						
Trial	Tap 1			Tap 3		
*****	A	B	r^2	A	B	r^2
1	218.33	1.71	0.97	206.26	1.69	0.99
2,3,4	155.53	2.02	0.96	168.89	1.87	0.97
5,6,7	192.36	1.76	0.98	213.18	1.64	0.99
FIELD	191.41	1.68	1.00	NA	NA	NA

r^2 = coefficient of determination, i.e., variance of data from the best fit curve - the closer to 1.0, the better the fit.

Table 8. - Comparison of field calibration to data - combined.

Field cal $Q(ft^3/s)$ /trial	% Difference in Q^* (Tap 1)			% Difference in Q^* (Tap 3)		
	1	2,3,4	5,6,7	1	2,3,4	5,6,7
500	+16	-1	+5	+8	-2	+9
1000	+18	+14	+8	+9	+6	+8
1500	+19	+24	+10	+9	+11	+7
2000	+20	+32	+12	+9	+15	+6
2500	+20	+38	+13	+9	+18	+6
3000	+21	+44	+14	+9	+20	+5

* "+" indicates actual flow is greater than field calibration.
"-" indicates actual flow is less than field calibration.

From the data, the following conclusions can be drawn:

- The data consistently plotted below the existing field calibration curve for all the trials and tended to fall farther away from the existing curve the more uneven the flow distribution (fig. 8). This result indicates that the actual discharge over the weir is more than the existing calibration curve predicts, and can sometimes vary as much as 50 pct for poor flow conditions and the existing tap location.

- The situations in which flow was through one of the control devices only (trials 1 to 4) showed the most variance in the data, and the best fit curves tended to be lower than the situations in which flow was equally distributed between the control devices (trials 5 to 7). The more equally distributed flow situations also tended to plot closer to the existing calibration curve, which would be expected. The situation in which the slide gate was closed and the butterfly valve was opened partially (trials 2 to 4) was the worst case. The butterfly valve tended to constrict flow and shoot it out at an angle, causing oscillating wave action and spraying over the outlet works side walls.

- Most of the data fall within a 10-pct margin of error for each best fit curve, indicating that the weir behaves consistently within most of the discharge ranges. Data became significantly more scattered for discharges greater than 2,500 ft^3/s, which was caused in part by flow restrictions at the control structures, as well as increased turbulence near the measurement device. The reliability of the calibration curves for the weir decreases significantly beyond 2,500 ft^3/s, and should be taken into consideration when determining discharges above this value.

- Data for tap 3 were significantly less scattered than data for tap 1 and tended to lie closer to the existing calibration curve. This result most likely occurred because at higher flows, the hydraulic jump tended to move downstream far enough to create a significant amount of wave action and turbulence near tap 1, making head measurements at tap 1 more varied. Tap 3 was located far enough downstream to avoid turbulence created by the jump, but still far enough upstream to avoid the effects of the weir. Flow near the left side of the weir is noticeably smoother than on the right side for all operating conditions tested.

RADIAL GATE FLOW ANALYSIS

Background

The Vallecito spillway has three 37-ft-long radial gates set atop the ogee crest that control flow released. Measuring flow through radial gates has been the subject of many physical and theoretical studies since the gate's inception by French engineer Poiree in 1853 (Rhone, 1958). Some 140+ years since its inception, many radial gate installations still defy applying strict theoretical solutions for computing discharge. The problem is harbored in the numerous variables affecting the discharge coefficient: gate opening, gate radius to trunion height, channel invert curvature, gate lip seal design, and downstream submergence. Of these parameters, gate opening and invert curvature are of predominant importance for the Vallecito spillway.

Two methods of determining discharge coefficients for the Vallecito spillway gates were investigated. First, a numerical calibration program developed by Buyalski (1983) for calibrating radial gates in canals was used. Second, work by Pomeroy (1942) was used in conjunction with model study data for other similar radial gate controlled spillways.

Method 1, Application of Canal Radial Gate Algorithms

For straight, flat-bottom channels, investigators have developed complex algorithms and numerical methods which can provide satisfactory results (Buyalski [1983], Metzler [1948]). However, radial gates used to control spillway flows such as at Vallecito often seat atop an ogee crest. The curvature of an ogee crest causes a nonhydrostatic pressure distribution downstream from the gate, which also affects the gate discharge coefficient. The canal radial gate algorithms do not account for the influence of an ogee crest. However, the Vallecito ogee is a suppressed overfall design, which means the shape of the ogee is much flatter than a zero pressure nappe design (fig. 16). Suppressed overfall designs were used on many Reclamation gated spillways. As pointed out in *Engineering Monograph 9* (Bradely, 1952), the flat ogee design was incorporated to accommodate the relatively flat downstream slopes of many embankment dams and/or to prevent low pressures on the spillway at small openings by matching the flow trajectory issuing from below the gate.

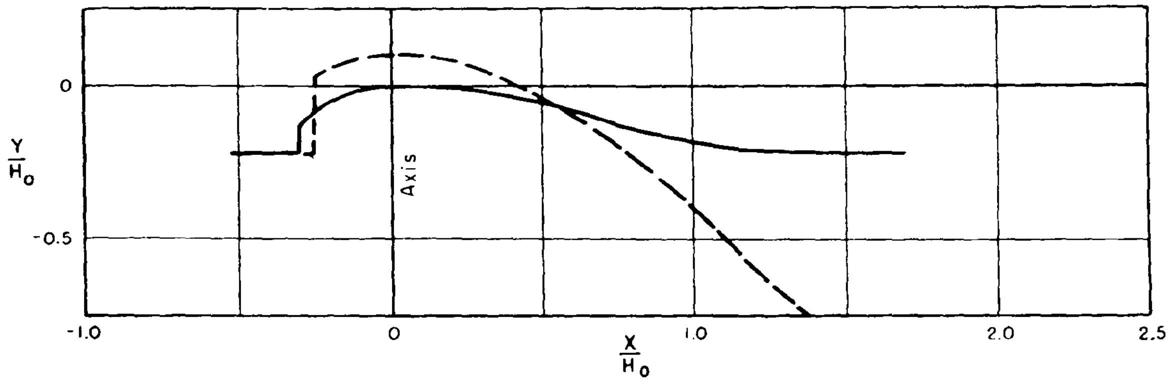
In the 1980s, Reclamation conducted an extensive research program to obtain a better definition of the discharge characteristics of canal radial gates. Algorithms were developed to represent a systematic method of illustrating the complete discharge characteristics of canal radial gates. The algorithms calculate the coefficient of discharge for submerged and free flow conditions through the gates, and accommodate a wide range of water levels and radial gate geometry normally encountered in Reclamation's design and construction of canal check gate structures. The algorithms were incorporated into the computer program RADGAT, which could be run on mainframe or workstation computer platforms.

Theory. - The general equation for discharge through an underflow gate can be obtained from Bernoulli's equation and is expressed as:

$$Q=C_d*GO*GW*(2*g*H)^{0.5} \tag{4}$$

where: Q = discharge
Cd = coefficient of discharge
GO = gate opening
GW = gate width
g = gravitational constant
H = head term

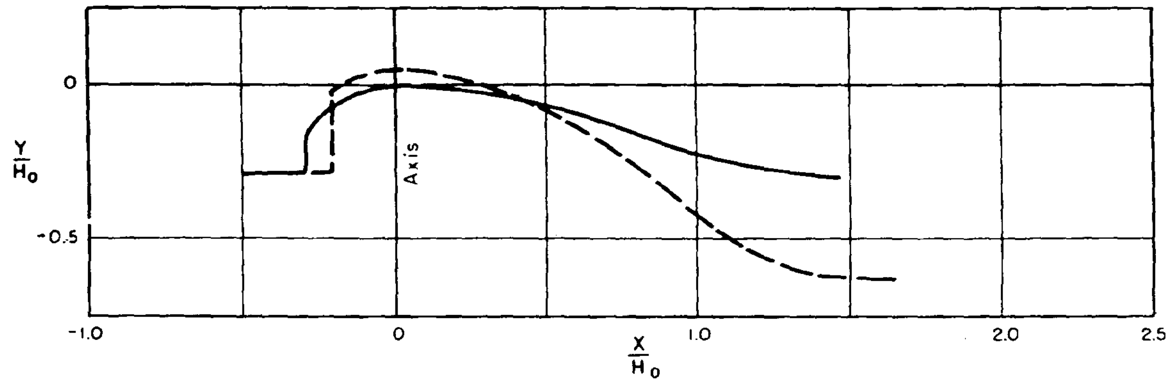
The definition of the head term is critical to the development of the coefficient of discharge, C_d. The coefficient of discharge varies significantly and has different characteristics depending on the definition used. Buyalski's algorithms use Metzler's concept, which defines the head term as the upstream depth measured from the gate sill to the upstream water surface. Metzler's concept compares the upstream and downstream depth-to-pinion height ratio to the coefficient of discharge. The three coordinates produce a "map" similar to a topographic map. This map represents the flow characteristics for a particular radial gate geometry, i.e., one gate lip seal design, one gate opening, and one gate arm radius over a range of flows. Each variation of gate geometry requires a new map. Numerous maps were generated from experimental data, and a family of curves was produced for a wide range of geometries. This method proved to be very accurate for radial gate installations conforming to standard canal radial gate installations.



GRANBY DAM

$\frac{H_0}{P+E} = \frac{5.0}{3.1}$ $C_M = 3.20$ ——— $C_D = 3.71$ - - -

APPENDIX - FIG. 8C

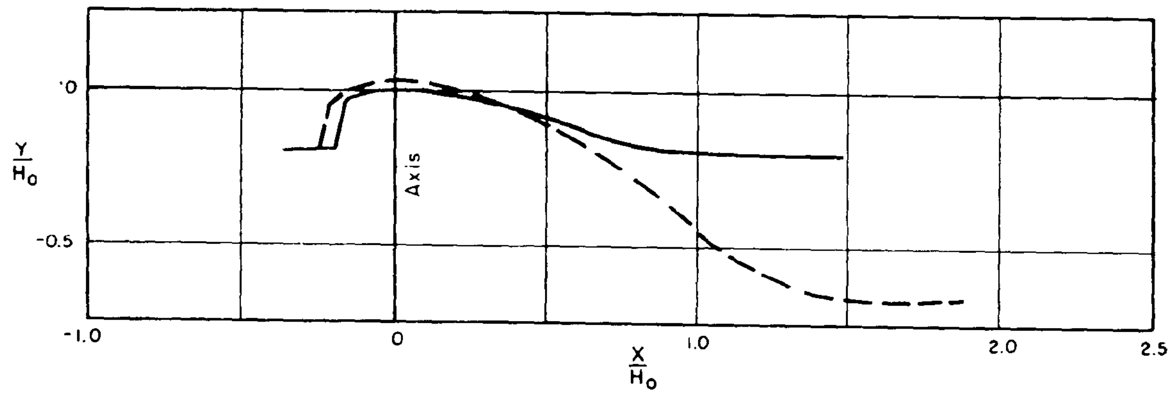


VALLECITO DAM

$\frac{H_0}{P+E} = \frac{3.8}{3.0}$ $C_M = 3.42$ ——— $C_D = 3.72$ - - -

CONTRACTION IN CHUTE D.S.

APPENDIX - FIG. 8F



BOYSEN DAM

(FINAL DESIGN)

$\frac{H_0}{P+E} = \frac{5.20}{4.51}$ $C_M = 3.45$ ——— $C_D = 3.57$ - - -

APPENDIX - FIG. 11C

Figure 16. - Comparison of overfall suppressed spillways, extracted from figure 24, *Engineering Monograph 9.*

The upstream and downstream water depths used in the discharge algorithms for the radial gates are based on a depth that would occur in a rectangular channel having the same width as the radial gate and the same invert elevation as the gate sill. Because this case does not usually exist in most designs, the energy balance equation and the Newton method of successive approximation are used to convert upstream water depth measurements in a non-rectangular section to an equivalent normal depth for a rectangular section, which accounts for head losses caused by the transitions. The same procedure is applied downstream from the gates. If free flow conditions exist downstream, only the upstream energy balance equation is solved.

The RADGAT program will compute gate openings required for a given discharge, or discharge for given gate openings. The program will also produce discharge rating tables, i.e., tables showing gate opening for ranges of upstream and downstream head over a range of incremental discharges. The program will solve for free or submerged conditions upstream and downstream from the gates, and can determine when these conditions exist given the initial parameters.

Application and Limitations. - The discharge algorithms apply primarily to canal radial gate check structures that are designed and constructed by Reclamation, and which have the following basic characteristics:

- Canal invert through the check structure must be nearly horizontal.
- Radius-to-pinion height ratio must be within the range of 1.2 to 1.7.
- The maximum upstream and downstream water depth-to-pinion height ratio must be less than 1.6.
- The algorithms are based on the hard-rubber-bar gate lip seal design. Correction algorithms were written for the music note gate lip design and the combined hard-rubber-bar/music-note design.
- The gate faceplate must be smooth.

For the Vallecito spillway radial gates:

- The gates seat on the flat apex of the ogee crest. The curvature of the ogee is relatively flat compared to a zero pressure profile.
- The radius-to-pinion height ratio is 1.5.
- The upstream water depth-to-pinion height ratio is 1.2.
- The gate lip seal is a music note design.
- The faceplates are smooth.

Therefore, the Vallecito radial gates deviate from the standard canal design only in the curvature of the invert. In addition, flow enters the gates via a transition from a trapezoidal entrance to a rectangular section upstream from the gates, which is similar to many canal structures. Flow downstream from the gates is always free flow because of the super-critical slope on the downstream spillway chute.

Analysis and Results. - The discharge predicted using Buyalski's(1983) canal gate program matched reasonably well with crest control (gates up operation) discharge data obtained from the original model study for low flows (fig. 17). With increasing discharge, predicted values from the program RADGAT increasingly underestimated the discharge when compared to the crest control model discharge data. Although it was hoped the effects of the ogee curvature would not be pronounced, these results are expected and are in keeping with the spillway design philosophy. At small gate openings, the spillway curvature downstream from the gate closely follows a free jet trajectory, and therefore has little influence on the discharge coefficient. Given the discharge error identified when discounting the spillway curvature, a different method to estimate spillway flows was needed.

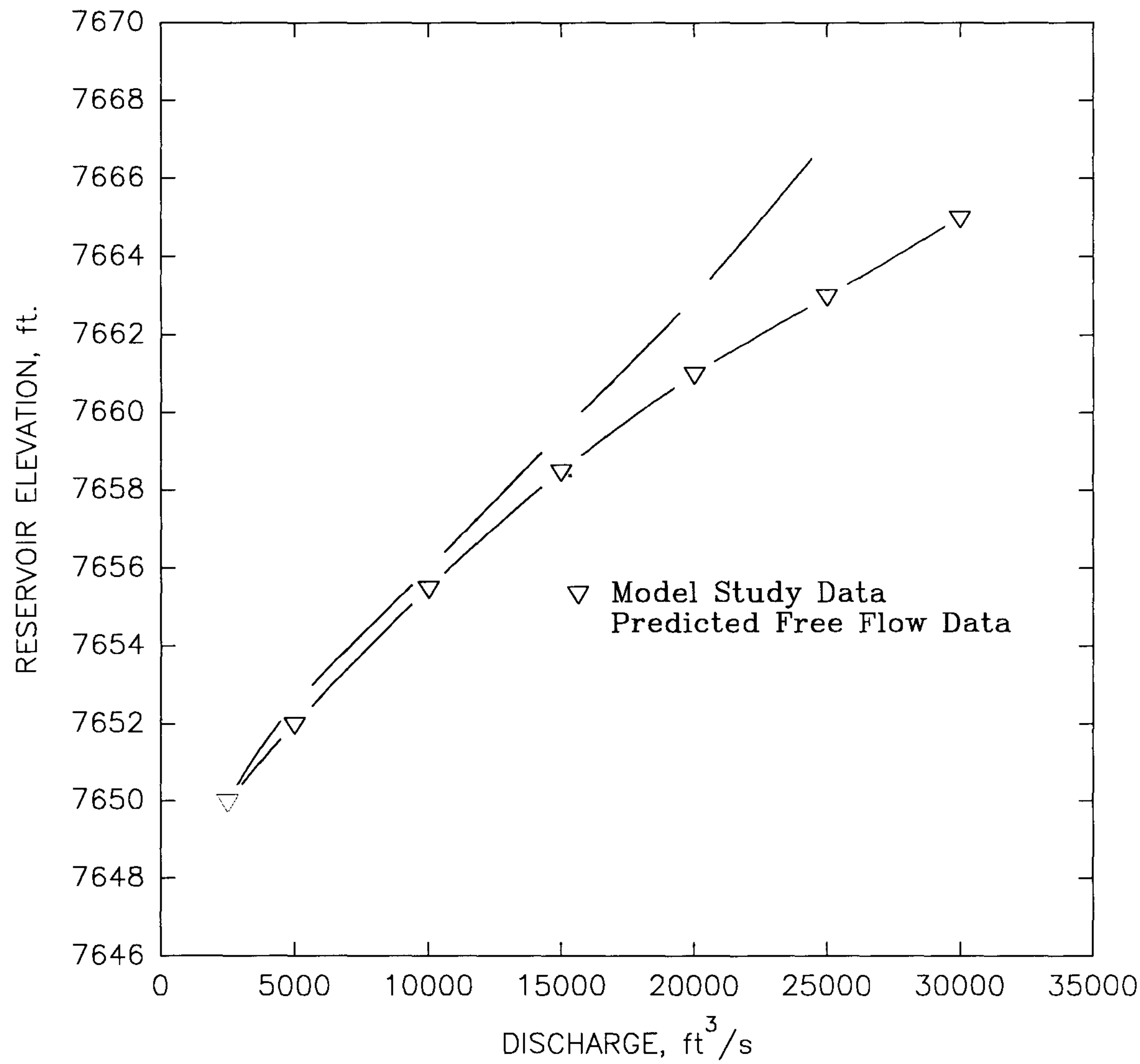


Figure 17. - Comparison between model study data and predicted values from program RADGAT for ungated spillway operation.

Method 2, Application of Model Data from Other Similar Spillways

An approach was developed which used the available model study data for crest control flow conditions at Vallecito Dam and gated operation model data for two similar style spillways at Granby and Boysen Dams.

Background. - Gate opening provides the greatest discharge coefficient uncertainty under conditions of small head (*h*) to gate opening (*d*) ratios. Rhone (1959) described the region of *h/d* below a value of 2.2 as notoriously difficult to evaluate. This difficulty is evident in data presented by Pomeroy (1942), figure 18, in which the coefficient of discharge varies widely between *h/d* values of about 1 (crest control) and 2.2 (orifice control). This zone of operation corresponds to a transition zone between weir control and orifice control. The transition zone is a region typically avoided in orifices designed for water measurement purposes. As a rule of thumb, Boss (1989) recommends orifices should not be used for water measurement under *h/d* ratios below 2. However, for radial gates, *h/d* ratios less than 2 are unavoidable. In addition to equation 4, a second common form of the discharge equation for a radial gate is:

$$Q=\frac{2}{3}C_d L\sqrt{2g}(h_1^{3/2}-h_2^{3/2}) \qquad (5)$$

where: C_d = coefficient of discharge
L = gate length
h_1 = depth of flow above the invert or spillway crest
h_2 = depth of flow above the gate lip
g = acceleration of gravity

Equations 4 and 5 are similar, except equation 5 resembles a weir equation with an exponent of 1.5 and equation 4 that of an orifice with an exponent of 0.5. Either form can be applied if appropriate coefficients are known from prototype or model test data.

Analysis and Results. - As presented by Pomeroy and discussed by Rhone, the greatest uncertainty in the coefficient of discharge occurs under low values of *h/d*. For the Vallecito spillway, this problem is reduced by the availability of model study data for crest control operation. Crest control flow data enable coefficient of discharge data to be calculated for the condition of $h/d \approx 1$. Crest control discharge coefficients were calculated using a design head, C_d, of $0.639(3.42/(2/3*(2g)^{0.5}))$ (fig. 16), and the relationship presented for other than design head given on figure 19. Coefficients were calculated to conform to equation 5.

Discharge coefficients were also calculated for gated operation at Boysen Dam and Granby Dam spillways. Both spillways are suppressed overfall ogee shapes similar to Vallecito (fig. 16), and have discharge ratings developed from model studies. Discharge coefficients were calculated for each spillway as a function of *h/d* and gate opening. Separate coefficient curves were calculated for constant gate openings to minimize the influence of orifice geometry. The effect of orifice geometry on the discharge coefficient is addressed in considerable depth by Buyalski and ignored by the simple approach of Pomeroy. Discharge coefficients for gate openings of 1, 2, 4, 6, 8, 10, 12, and 16 ft were calculated. Spillway crest control flow data for Vallecito and the spillway gate control data for Granby and Boysen were then used to estimate the discharge coefficients for the Vallecito Dam spillway. Using the crest control flow data from the Vallecito model tests provided a known starting coefficient at small values of *h/d* for each gate setting.

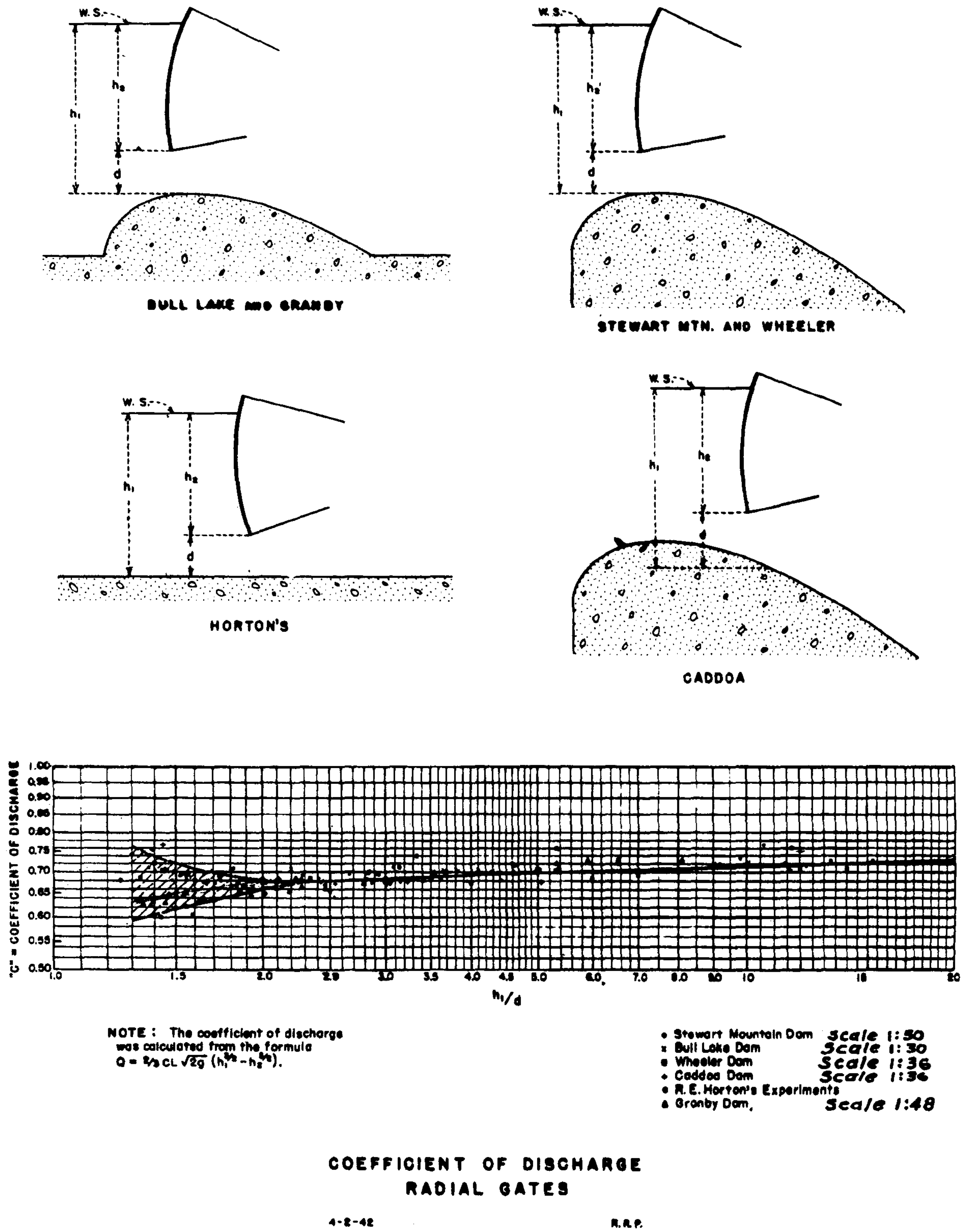
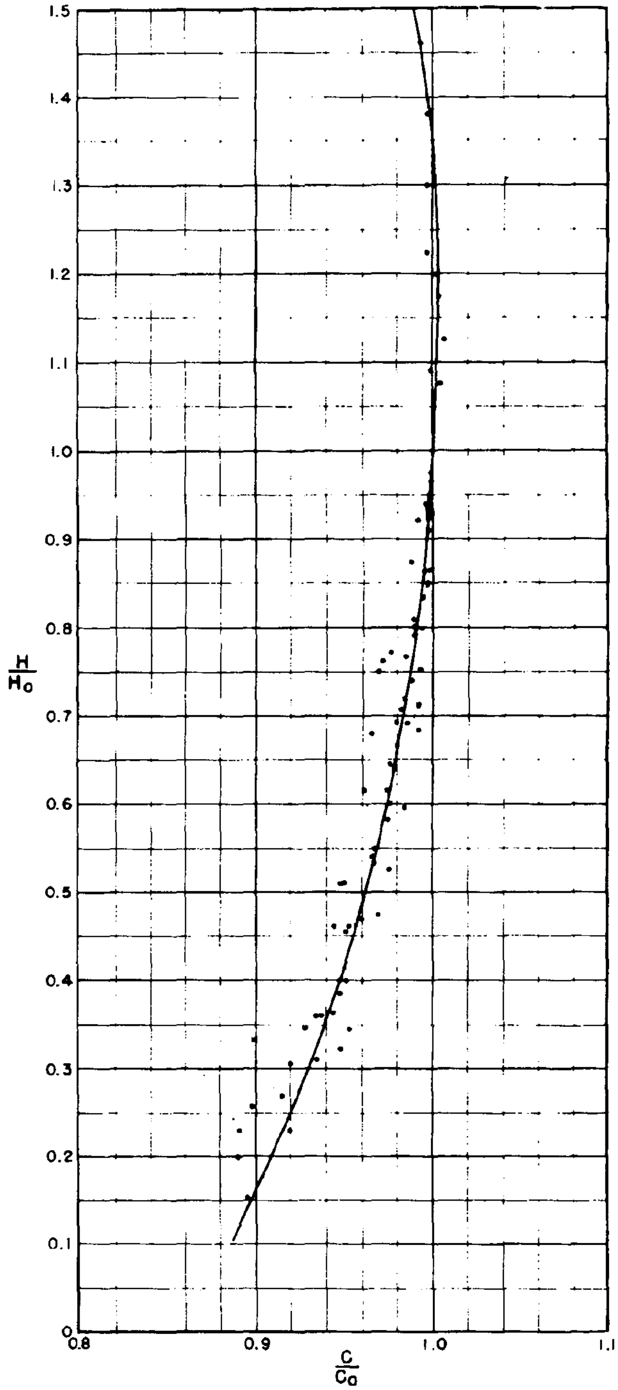
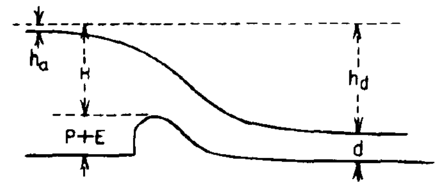


Figure 18. - Coefficient of discharge data for radial gate controlled spillways given by Pomeroy.





$\frac{H_o}{P+E}$	SPILLWAY	$\frac{h_d+d \text{ (Actual)}}{h_d+d \text{ (Experimental)}}$
17	Rye Patch	0.71
8	Alcova	.71
5.2	Boysen (Final)	.72
5.5	Green Mountain	.75
3.1	Scofield	.76
3.6	Bartlett	.76
4.8	Anderson Ranch	.76
5.0	Cachuma	.78
3.4	Falcon	.79
2.8	Unity	.79
2.8	Dickinson	.80
4.5	Medicine Creek (Low)	.80
2.4	Boca	.83
1.3	Fresno	.84
3.7	Boysen (Prelim.)	.85
2.6	Caballo	.86

Figure 19. - Coefficient of discharge for other than design head on overfall suppressed spillways. Extracted from *Engineering Monograph 9*, figure 35.

The data for each gate opening was then fit by regression analysis to an equation of the form:

$$C_d = a + b e^{-h/d} \tag{6}$$

where a and b are coefficients and e is the natural logarithm base, 2.718.

After fitting all data sets to equation 6, an inspection of the coefficients found coefficient a was nearly constant, with an average value of 0.713. Following this observation, a second regression of the data was conducted with coefficient a held constant at 0.713 (fig. 20). Coefficient b was found to vary as a linear function of gate opening, d, in ft, and could be expressed as:

$$b = -0.405 + 0.011 * d \tag{7}$$

Using equations 5, 6, and 7, the discharge for any gate opening and upstream head can be estimated for the Vallecito spillway. Figure 21 shows spillway discharge ratings produced using equations 5, 6, and 7. Each gate opening curve shown indicates total spillway discharge for equal gate operation. Discharge for single or double gate operation can be estimated by subsituting the appropriate gate length into equation 5. The minimum discharge predicted for each gate opening represents the crest control discharge for the given head. Crest control is given by the equations for conditions of h/d = 1. An example of computing spillway discharge is:

Given: Reservoir elevation = 7660.0 ft, head on the spillway = 14.0 ft
Gate opening = 2.0 ft (gate length = 111.0 ft [three 37-ft gates])

Using equation 7, Beta = -0.405+0.011*2.0
= -0.383

Subsituting -0.383 in equation 6 gives:

$$C_d = 0.713 - 0.383 * e^{-14/2}$$
$$= 0.7127$$

Solving equation 5 for discharge gives:

$$Q = 0.7127 * 2/3 * 8.025 * 111.0 * (14^{1.5} - 12^{1.5})$$
$$= 4{,}576 \text{ ft}^3/\text{s}$$

To evaluate the likely error in calculating the coefficient of discharge using equations 6 and 7, the percent deviation in the predicted C_d value from the actual value was determined for all data used: Granby spillway discharge data, Boysen spillway discharge data, and Vallecito spillway crest control flow data. The maximum deviation for the data is ±3 pct. The discharge measurement accuracy expected by applying this method to Vallecito spillway is estimated to be about ±5 pct given the deviation in C_d values presented by Pomeroy, the use of crest control model data to reduce the uncertainty at low values of h/d, and the geometric similarity of Granby and Boysen spillways. Spillway discharge rating tables generated using equations 5, 6, and 7 are given in table 9.

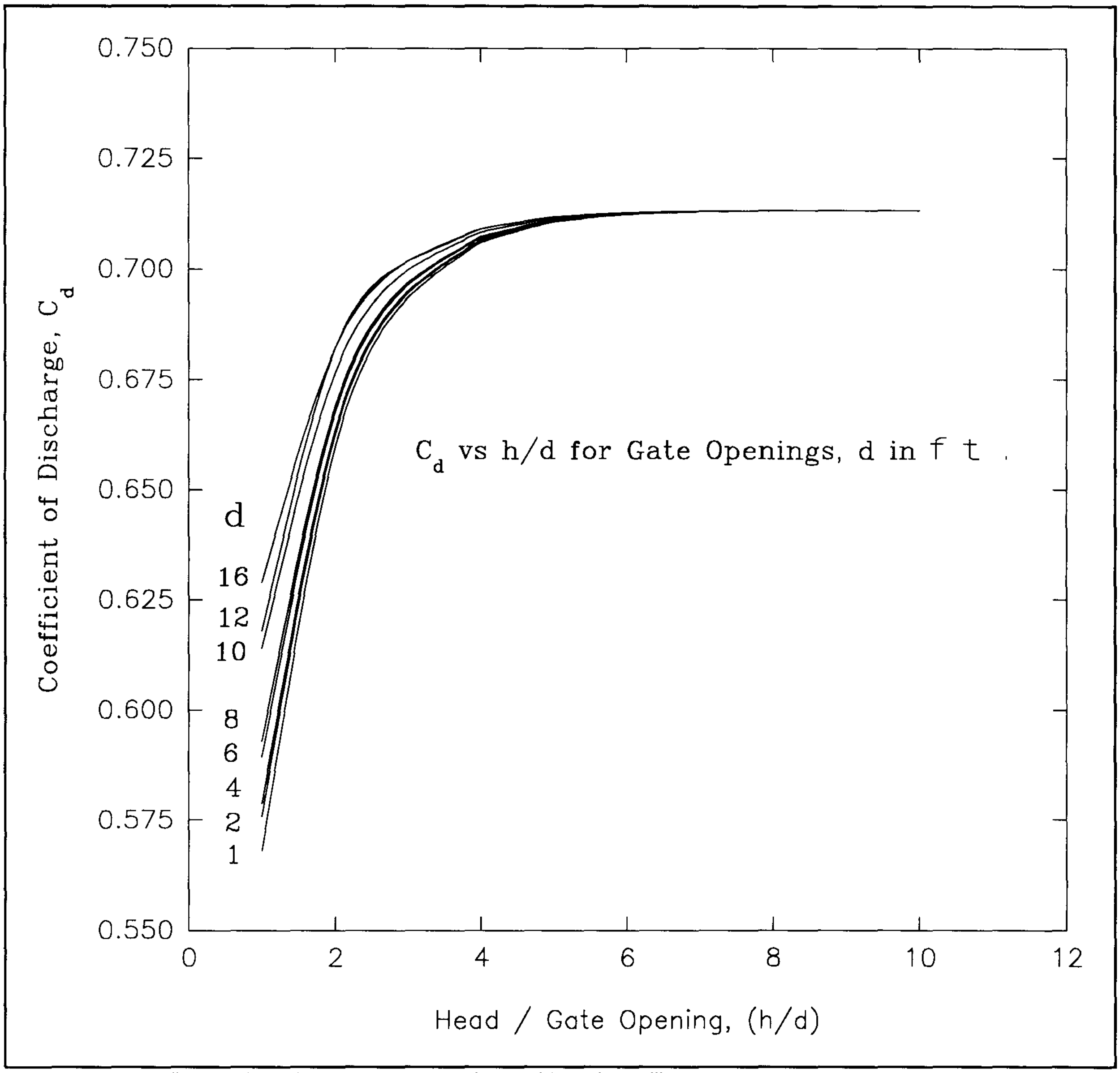


Figure 20. - Coefficient of discharge, C_d, versus h/d for Vallecito spillway.

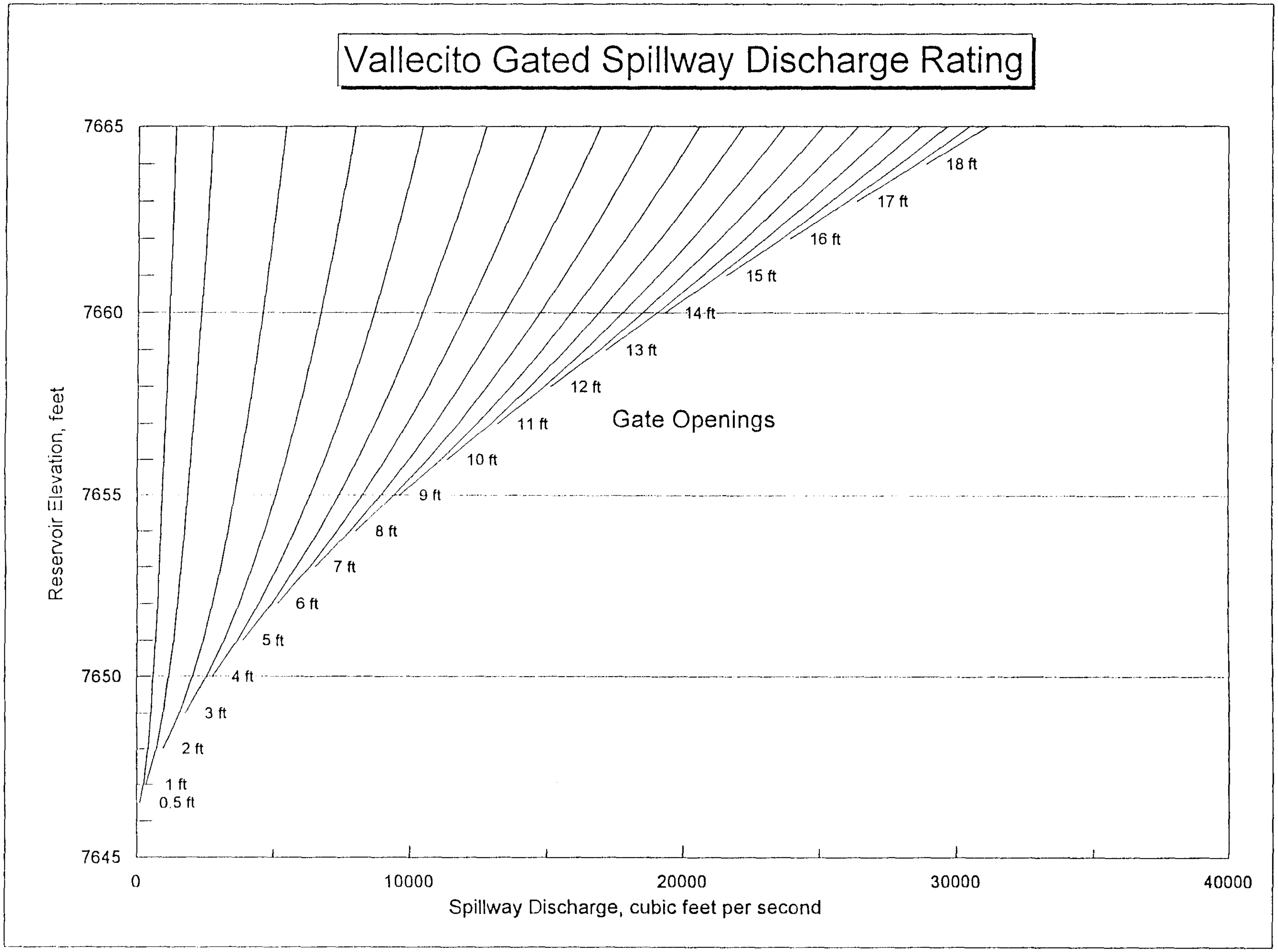


Figure 21. - Vallecito spillway rating curves.

Table 9. - Vallecito spillway discharge ratings.

Discharge (cfs)	Reservoir Elevation (ft)	Gate Opening (ft)
200.00	7646.71	Free Flow
	7647.21	0.31
	7647.71	0.25
400.00	7647.12	Free Flow
	7647.62	0.56
	7648.12	0.46
	7648.62	0.41
600.00	7647.47	Free Flow
	7647.97	0.79
	7648.47	0.65
	7648.97	0.58
	7649.47	0.53
	7649.97	0.49
	7650.47	0.46
	7650.97	0.43
800.00	7647.78	Free Flow
	7648.28	1.01
	7648.78	0.84
	7649.28	0.75
	7649.78	0.68
	7650.28	0.63
	7650.78	0.60
	7651.28	0.56
	7651.78	0.54
	7652.28	0.51
	7652.78	0.49
	7653.28	0.47
	7653.78	0.46
	7654.28	0.44
	7654.78	0.43
	7655.28	0.42
	7655.78	0.41
	7656.28	0.40

Table 9. - Vallecito spillway discharge ratings (continued).

Discharge (cfs)	Reservoir Elevation (ft)	Gate Opening (ft)
1000.00	7648.06	Free Flow
	7649.06	1.02
	7650.05	0.83
	7651.05	0.73
	7652.05	0.66
	7653.05	0.61
	7654.05	0.57
	7655.05	0.53
	7656.06	0.50
	7657.06	0.48
	7658.06	0.46
	7659.06	0.44
	7660.06	0.42
	7661.06	0.41
	7662.06	0.40
	7663.06	0.38
	7664.06	0.37
1500.00	7648.68	Free Flow
	7649.68	1.44
	7650.68	1.19
	7651.68	1.05
	7652.68	0.95
	7653.68	0.88
	7654.68	0.83
	7655.68	0.78
	7656.68	0.74
	7657.68	0.70
	7658.68	0.67
	7659.68	0.65
	7660.68	0.63
	7661.68	0.60
	7662.68	0.59
	7663.68	0.57
	7664.68	0.55
2000.00	7649.24	Free Flow
	7650.24	1.84
	7651.24	1.52
	7652.24	1.35
	7653.24	1.23
	7654.24	1.14
	7655.24	1.07
	7656.24	1.01
	7657.24	0.96
	7658.24	0.92
	7659.24	0.88
	7660.24	0.85

Table 9. - Vallecito spillway discharge ratings (continued).

Discharge (cfs)	Reservoir Elevation (ft)	Gate Opening (ft)
2000.00	7661.24	0.82
	7662.24	0.79
	7663.24	0.77
	7664.24	0.75
2500.00	7649.75	Free Flow
	7650.75	2.22
	7651.75	1.85
	7652.75	1.64
	7653.75	1.50
	7654.75	1.40
	7655.75	1.31
	7656.75	1.24
	7657.75	1.18
	7658.75	1.13
	7659.75	1.09
	7660.75	1.05
	7661.75	1.01
	7662.75	0.98
	7663.75	0.95
	7664.75	0.92
3000.00	7650.23	Free Flow
	7651.23	2.58
	7652.23	2.16
	7653.23	1.92
	7654.23	1.76
	7655.23	1.64
	7656.23	1.54
	7657.23	1.47
	7658.23	1.40
	7659.23	1.34
	7660.23	1.29
	7661.23	1.24
	7662.23	1.20
	7663.23	1.16
	7664.23	1.13
3500.00	7650.67	Free Flow
	7651.67	2.94
	7652.67	2.47
	7653.67	2.20
	7654.67	2.02
	7655.67	1.88
	7656.67	1.77
	7657.67	1.68
	7658.67	1.61
	7659.67	1.54

Table 9. - Vallecito spillway discharge ratings (continued).

Discharge (cfs)	Reservoir Elevation (ft)	Gate Opening (ft)
3500.00	7660.67	1.48
	7661.67	1.43
	7662.67	1.38
	7663.67	1.34
	7664.67	1.30
4000.00	7651.10	Free Flow
	7652.10	3.28
	7653.10	2.76
	7654.10	2.47
	7655.10	2.27
	7656.10	2.11
	7657.10	2.00
	7658.10	1.90
	7659.10	1.81
	7660.10	1.74
	7661.10	1.68
	7662.10	1.62
	7663.10	1.57
	7664.10	1.52
4500.00	7651.50	Free Flow
	7652.50	3.61
	7653.50	3.06
	7654.50	2.73
	7655.50	2.51
	7656.50	2.35
	7657.50	2.22
	7658.50	2.11
	7659.50	2.02
	7660.50	1.93
	7661.50	1.86
	7662.50	1.80
	7663.50	1.75
	7664.50	1.69
5000.00	7651.89	Free Flow
	7652.89	3.93
	7653.89	3.34
	7654.89	2.99
	7655.89	2.75
	7656.89	2.57
	7657.89	2.43
	7658.89	2.31
	7659.89	2.21
	7660.89	2.13
	7661.89	2.05
	7662.89	1.98

Table 9. - Vallecito spillway discharge ratings (continued).

Discharge (cfs)	Reservoir Elevation (ft)	Gate Opening (ft)
5000.00	7663.89	1.92
	7664.89	1.87
5500.00	7652.27	Free Flow
	7653.27	4.25
	7654.27	3.62
	7655.27	3.25
	7656.27	2.99
	7657.27	2.80
	7658.27	2.64
	7659.27	2.52
	7660.27	2.41
	7661.27	2.32
	7662.27	2.24
	7663.27	2.16
	7664.27	2.10
6000.00	7652.63	Free Flow
	7653.63	4.56
	7654.63	3.90
	7655.63	3.50
	7656.63	3.23
	7657.63	3.02
	7658.63	2.85
	7659.63	2.72
	7660.63	2.60
	7661.63	2.51
	7662.63	2.42
	7663.63	2.34
	7664.63	2.27
6500.00	7652.98	Free Flow
	7653.98	4.86
	7654.98	4.17
	7655.98	3.75
	7656.98	3.46
	7657.98	3.24
	7658.98	3.06
	7659.98	2.92
	7660.98	2.80
	7661.98	2.69
	7662.98	2.60
	7663.98	2.52
	7664.98	2.44
7000.00	7653.32	Free Flow
	7654.32	5.16
	7655.32	4.44

Table 9. - Vallecito spillway discharge ratings (continued).

Discharge (cfs)	Reservoir Elevation (ft)	Gate Opening (ft)
7000.00	7656.32	4.00
	7657.32	3.69
	7658.32	3.45
	7659.32	3.27
	7660.32	3.12
	7661.32	2.99
	7662.32	2.88
	7663.32	2.78
	7664.32	2.69
7500.00	7653.66	Free Flow
	7654.66	5.45
	7655.66	4.71
	7656.66	4.24
	7657.66	3.92
	7658.66	3.67
	7659.66	3.47
	7660.66	3.31
	7661.66	3.18
	7662.66	3.06
	7663.66	2.95
	7664.66	2.86
8000.00	7653.98	Free Flow
	7654.98	5.74
	7655.98	4.97
	7656.98	4.48
	7657.98	4.14
	7658.98	3.88
	7659.98	3.68
	7660.98	3.51
	7661.98	3.36
	7662.98	3.24
	7663.98	3.13
	7664.98	3.03
8500.00	7654.30	Free Flow
	7655.30	6.02
	7656.30	5.23
	7657.30	4.72
	7658.30	4.37
	7659.30	4.09
	7660.30	3.88
	7661.30	3.70
	7662.30	3.55
	7663.30	3.42
	7664.30	3.30

Table 9. - Vallecito spillway discharge ratings (continued).

Discharge (cfs)	Reservoir Elevation (ft)	Gate Opening (ft)
9000.00	7654.61	Free Flow
	7655.61	6.30
	7656.61	5.48
	7657.61	4.96
	7658.61	4.59
	7659.61	4.30
	7660.61	4.08
	7661.61	3.89
	7662.61	3.73
	7663.61	3.60
	7664.61	3.48
9500.00	7654.91	Free Flow
	7655.91	6.57
	7656.91	5.73
	7657.91	5.19
	7658.91	4.81
	7659.91	4.51
	7660.91	4.27
	7661.91	4.08
	7662.91	3.91
	7663.91	3.77
	7664.91	3.65
10000.00	7655.21	Free Flow
	7656.21	6.85
	7657.21	5.98
	7658.21	5.43
	7659.21	5.02
	7660.21	4.72
	7661.21	4.47
	7662.21	4.27
	7663.21	4.10
	7664.21	3.95

Table 9. - Vallecito spillway discharge ratings (continued).

Discharge (cfs)	Reservoir Elevation (ft)	Gate Opening (ft)
11000.00	7655.78	Free Flow
	7656.78	7.38
	7657.78	6.48
	7658.78	5.88
	7659.78	5.46
	7660.78	5.12
	7661.78	4.86
	7662.78	4.64
	7663.78	4.46
	7664.78	4.30
12000.00	7656.34	Free Flow
	7657.34	7.90
	7658.34	6.95
	7659.34	6.33
	7660.34	5.88
	7661.34	5.53
	7662.34	5.24
	7663.34	5.01
	7664.34	4.81
13000.00	7656.88	Free Flow
	7657.88	8.40
	7658.88	7.43
	7659.88	6.78
	7660.88	6.30
	7661.88	5.92
	7662.88	5.62
	7663.88	5.37
	7664.88	5.16
14000.00	7657.41	Free Flow
	7658.41	8.90
	7659.41	7.89
	7660.41	7.21
	7661.41	6.71
	7662.41	6.32
	7663.41	6.00
	7664.41	5.73
15000.00	7657.91	Free Flow
	7658.91	9.39
	7659.91	8.35
	7660.91	7.64
	7661.91	7.12
	7662.91	6.71
	7663.91	6.37
	7664.91	6.09

Table 9. - Vallecito spillway discharge ratings (continued).

Discharge (cfs)	Reservoir Elevation (ft)	Gate Opening (ft)
16000.00	7658.40	Free Flow
	7659.41	9.86
	7660.41	8.79
	7661.41	8.07
	7662.41	7.52
	7663.41	7.09
	7664.41	6.74
17000.00	7658.89	Free Flow
	7659.89	10.33
	7660.89	9.23
	7661.89	8.49
	7662.89	7.92
	7663.89	7.47
	7664.89	7.11
18000.00	7659.37	Free Flow
	7660.37	10.78
	7661.37	9.67
	7662.37	8.90
	7663.37	8.31
	7664.37	7.85
19000.00	7659.83	Free Flow
	7660.83	11.23
	7661.83	10.09
	7662.83	9.30
	7663.83	8.70
	7664.83	8.22
20000.00	7660.28	Free Flow
	7661.28	11.68
	7662.28	10.52
	7663.28	9.71
	7664.28	9.09
21000.00	7660.72	Free Flow
	7661.72	12.11
	7662.72	10.93
	7663.72	10.10
	7664.72	9.47
22000.00	7661.16	Free Flow
	7662.16	12.54
	7663.16	11.34
	7664.16	10.50

Table 9. - Vallecito spillway discharge ratings (continued).

Discharge (cfs)	Reservoir Elevation (ft)	Gate Opening (ft)
23000.00	7661.58	Free Flow
	7662.58	12.96
	7663.58	11.75
	7664.58	10.89
24000.00	7662.00	Free Flow
	7663.00	13.38
	7664.00	12.14
25000.00	7662.41	Free Flow
	7663.41	13.79
	7664.41	12.54
26000.00	7662.82	Free Flow
	7663.82	14.19
	7664.82	12.93
27000.00	7663.22	Free Flow
	7664.22	14.59
28000.00	7663.61	Free Flow
	7664.61	14.99
29000.00	7663.99	Free Flow
	7664.99	15.38
30000.00	7664.37	Free Flow

BIBLIOGRAPHY

Beichley, G.L. and Wagner, W.E., *Hydraulic Model Studies of Boysen Dam Spillway*, Hydraulic Laboratory Report No. 212, 1952.

Boss, M.C., *Discharge Measurement Structures*, International Institute for Land Reclamation and Improvement/ILRI, Third Edition, 1989.

Bradely, J.W., "Discharge Coefficients for Irregular Overfall Spillways," Bureau of Reclamation *Engineering Monograph 9*, Denver, Colorado, March, 1952.

Bureau of Reclamation, *Vallecito Dam SEED Data Book - Volume I.*

Buyalski, Clark P., *Discharge Algorithms for Canal Radial Gates*, Bureau of Reclamation report REC-ERC-83-9, December 1983.

Metzler, D.E., "A Model Study of Tainter Gate Operation," M.S. thesis, State University of Iowa, Iowa City, Iowa, August, 1948.

Pomeroy, R.R., *Hydraulic Model Studies of Granby Dam Spillway, Colorado Big Thompson Project*, Hydraulic Laboratory Report No. 109, 1942.

Rhone, T.J., "Problems Concerning Use of Low Head Radial Gates", *Journal of Hydraulics*, American Society of Civil Engineers, HY2, February, 1959.

Mission

The mission of the Bureau of Reclamation is to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American Public.